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OLDMAN RIVER DAM

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DESIGN REPORT

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OLDMAN RIVER DAM

DESIGN REPORT

Prepared for:

ALBERTA PUBLIC WORKS, SUPPLY AND SERVICES

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OLDMAN RIVER DAM
DESIGN REPORT

SUMMARY

General

This report presents the design for the works that are to be constructed under the following eight contracts:

- Dam Contract.
- Dyke Contract.
- Drainage Tunnels Contract.
- Road Embankment No. 1 Contract.
- Road Embankment Nos. 2 and 3 Contract.
- Riprap Supply Contract
- Road Surfacing Contract.
- Instrumentation Contract.

The report updates the design work recorded in the Preliminary Engineering Report and describes the final design of the project works.

The project works primarily consist of:

- the construction of the dam and dyke from soil and rock excavated from the spillway excavation and obtained from sources in the vicinity of the project site,
- the spillway excavation
- the construction of a grout curtain across the river valley and into both river banks to reduce seepage beneath and around the dam and spillway,
- the drainage tunnels in both river banks, one of which will also provide access to the valve chamber,

- the system of pressure relief drains leading to the drainage tunnels to control groundwater pressures beneath the spillway headworks and in the excavated and existing slopes,
- five road embankments which will use fill that must be excavated for the spillway, and
- instrumentation to measure the performance of the dam.

The dam and other works have been designed using state-of-the-art analytical methods and to criteria adopted by engineers and owner authorities in Canada and around the world. References in the public domain that attest to this precedence are cited in the main body of the report and its appendices.

Dam

The 76 m high dam will be a zoned embankment structure with a wide central core constructed of clay till, protected on both sides with a sand filter zone. On the downstream side, a gravel chimney drain will collect seepage through the core and will lead it down to a drainage blanket beneath the downstream shell of the dam. Rock from the required excavations will be placed in the downstream shell and in part of the upstream shell. A thick pervious layer of sand and gravel will stabilize the upstream shell during rapid drawdown of the reservoir. Protection of the upstream face of the dam against wave action will be provided by a layer of quarried rock riprap. A similar riprap zone will prevent erosion of the downstream toe by high velocity back eddies during spillway operation. The dam, including the 500 m long freeboard section on the left river bank, will contain approximately 8,200,000 m^3 of fill. Of this volume approximately 3,600,000 m^3 will come from the spillway excavation.

Geology and Foundation Conditions

The overburden in the dam foundation comprises glaciolacustrine silt and clay and clay till on the river banks and pervious sand and gravel alluvium in the river bottom. Loose colluvium interfingers with the alluvium at the bottom of the river banks.

A massive 37 m thick predominantly sandstone layer underlies the river bottom alluvium. A sequence of interbedded sandstone, siltstone and claystone overlies the basal sandstone in the river banks. The basal sandstone forms the lowest part of the Porcupine Hills formation. The middle section of the basal sandstone, although predominantly sandstone, includes siltstone and claystone beds. This section begins at a depth of some 15 m below the surface of the basal sandstone and contains discontinuities or seams generally following the sub-horizontal bedding planes. The seams control the stability, and dictate the shape and size of the dam. The strength and continuity of the seams have been extensively investigated by 3300 m of bedrock coring, by logging surface outcrops and exposures in three exploratory shafts and in a test tunnel and by laboratory shear box testing of 223 undisturbed samples of intact rock and seam material. Some of the seams are found to be continuous over distances large enough to be significant to the design. The actual strength along the seams will be less than that of the adjacent intact rock but greater than the minimum residual strength measured on laboratory shear strength samples. Considerable effort was concentrated on determining the insitu state of the material in the seams and in the adjacent rock. A representative shear strength was assigned to each of the major seams and to the rock in the remainder of the structure foundation. These were used in the dam and other stability analyses. The representative shear strengths were determined as the sum of the laboratory-determined residual shear strength and a geological component which takes into account the roughness, thicknesses, and in situ state of the seams.

The interbedded sandstone, siltsone, and claystone sequence overlying the basal sandstone in both river banks also contains similar seams, some of which appear to be continuous. These have also been mapped and tested and appropriate shear strength parameters have been introduced into the design of the existing and excavated slopes in the spillway excavation and in the dam abutments.

The magnitude and variation of the laboratory-determined residual shear strength measured in the seam samples was found to be consistent with the plasticity, clay content and mineralogy of the mudrocks.

No evidence of bentonite or bentonitic material of any kind was found in the upper 70m of the Willow Creek formation that was drilled at the site nor was bentonitic material found in any of the other formations. Bentonite and bentonitic material have been reported by Douglas (1950) to be present in a few isolated thin layers towards the base of the 840 m thick Willow Creek formation. The top of the Willow Creek formation lies at the base of the basal sandstone at the site at a depth of about 50 m below the river bottom.

Stability Analyses

The dam will have overall slopes of approximately four horizontal to one vertical to provide stability against sliding along surfaces passing down through the core and horizontally along the seams that are known to exist in the foundation rock. Representative values for the angle of shearing resistance of the seam were determined and used in the design. The seams within the basal sandstone were found to control the stability of the dam as a substantial portion of all critical slip surfaces passed through them. The stability analyses considered various critical conditions that could occur during the life of the project. These included the conditions that would exist after steady state seepage has developed through the dam with the resevoir at full supply level, during rapid drawdown of the reservoir, during a seismic event and immediately after construction. Earthquake records in the geological region were analyzed to determine the magnitude of the seismic acceleration that would result from the maximum credible earthquake occurring at the site. Pore water pressure parameters were selected based on the laboratory triaxial test data and insitu data from other dams of the same type and on similar foundations. Both limit equilibrium and finite element analyses of the stability of the dam under static and dynamic loading conditions have been done. It was calculated that deformation of the dam following the maximum credible earthquake would be well within generally accepted limits. The dam will be stable under all operating and loading conditions with safety factors equal to or greater than those generally considered a minimum for a dam of this size.

Seepage Control

Seepage beneath and around the dam and spillway will be reduced and controlled by a clay core cutoff through the pervious alluvium beneath the dam, by a grout curtain across the river valley and into both river banks and by a system of pressure relief drains into a drainage tunnel through each river bank. The cutoff will be located in a trench excavated through the alluvial sands and gravels and through the claystone layer in the river bottom to the top of the basal sandstone. A two line grout curtain will be constructed from the bottom of the core trench down to the bottom of the basal sandstone. The grout holes will be inclined to intersect the vertical and horizontal joints. Similarly inclined and lined pressure relief holes will be drilled from the drainage tunnels sufficiently far downstream of the grout curtain to prevent particle migration under high hydraulic gradients.

Dewatering

The core trench and dam foundations will be excavated and prepared and the dam will be constructed in the dry behind cofferdams while the river is diverted through two 6.5 m diameter diversion tunnels driven through the left river bank. The cofferdams will also have a grout curtain to seal a pervious zone found during the site investigation to be associated with the upper structural unit in the basal sandstone in the centre of the river valley. The upstream cofferdam will be incorporated into the upstream toe of the dam. Both cofferdams have been designed for the 1:50 year flood level.

Foundation Excavation and Treatment

Loose and potentially liquefiable colluvium and alluvium will be removed from the dam foundation. Soil foundations at the top of the river bank will be excavated back to stable slopes. In the core contact area on the river banks the rock will be presheared and excavated to slopes not steeper than one horizontal to one vertical. After excavation and before grouting starts, the excavated rock surface of the core contract area will be scaled, trimmed, cleaned with air jets, shaped with concrete and protected with a layer of shotcrete.

Dam Fill Materials

The core will be constructed of approximately 1,500,000 m^3 of clay till obtained from the spillway excavation and from a borrow source upstream of the dyke. Moisture conditioning of the clay will be necessary generally to increase the moisture content. The specifications for moisture content, placement, compaction and required compaction equipment have been based on full scale test fills of clay till, glaciolacustrine clay and excavated rock.

Crushing, screening, washing, classifying and blending the naturally occurring sand and gravel alluvium found in the vicinity of the site will be required to produce filter, drain and riprap bedding materials meeting the specified gradations. An approximate total of 800,000 m^3 of these processed fill materials will be required. A trial processing program was carried out to demonstrate that the specified gradations can be achieved.

Approximately 3,000,000 m^3 of rock and soil excavated from the spillway and diversion tunnel inlet excavations will be used in the dam shells. The rock will be specified to have a maximum size of 300 mm. A narrow oversize zone will be placed on the downstream face of the dam for excavated rock up to 1 m size. The specified maximum size for rock fill will necessitate blasting in the excavation.

Granular fill for the upstream pervious zone and for the cofferdams will be obtained from the same sources as the filter drain and riprap bedding material. It is anticipated that most of the 2,500,000 m^3 of granular fill material that will be required will be obtained pitrun and will not require processing.

Four classes of riprap with a total volume of 200,000 m^3 will be specified for the dam to be underlain by 120,000 m^3 of bedding material. An additional 40,000 m^3 will be used to protect the left river banks upstream of the dam and dyke. The amount of slope protection varies according to the exposure of the slope, fetch and the probable periods of time that the reservoir will be at any particular level. The riprap below full supply level has been designed for a combined probability of maximum windspeed and reservoir level for a 1:100 year event. The riprap will be quarried and hauled to the project site under the Riprap Supply contract. Sources of suitable rock for riprap have been located between 35 km and 60 km to the west of the project site.

Freeboard

The calculated freeboard required for wave runup and wind setup is 2.2 m above full supply level. A total freeboard of 6.9 m will be provided above full supply level to allow the reservoir to surcharge during floods larger than the 1:1000 year event. A nominal freeboard of 1 m has been provided during the probable maximum flood.

Settlement

It has been estimated that settlement of the dam and its foundations will be in the order of 1 m where the height of fill is maximum. The crest of the dam and the dyke will be constructed with a longitudinal camber above the design level to compensate for the long term settlement.

Dyke and Freeboard Section of the Dam

The dyke and the freeboard section of the dam on the left river bank will have a total length of 2.1 km. The maximum height of these embankments will be 10 m at a section of the dyke where the depth of water at full supply level will be not greater than 3.1 m. Approximately 1.6 km of the dam and dyke will be normally above full supply level. The total volume of the dyke will be approximately 360,000 m^3. The dyke and freeboard section of the dam will be constructed primarily of 230,000 m^3 of glaciolacustrine clay removed from the borrow area to expose clay till required to complete the dam core. Approximately forty percent of the glaciolacustrine soils stripped from the surface of the clay till borrow pit will not be suitable, or will not be required, for construction and will have to be wasted. Slopes of four horizontal to one vertical have been used for both upstream and downstream slopes. A blanket drain will be placed beneath the downstream part of the section of the dyke that is higher than 6.5 m above excavation foundation. The upstream slopes will be protected with 40,000 m^3 of riprap. The downstream slopes will be covered with topsoil and will be seeded.

Construction Schedule

Five of the eight contracts will be awarded late 1987 and early 1988. During the following winter some of the spillway excavation will be started and the right bank drainage tunnel will be constructed to permit early installation of instrumentation to measure the progress of rebound of the spillway excavation. The river will be diverted commencing July 1988. The upstream cofferdam will be raised to elevation 1080 m before the Spring of 1989. During 1988 the core trench will be excavated, the dam shell foundations will be stripped and the grout curtain will be started. Some construction of the dam shells can be achieved and the left bank drainage tunnel and the dyke can be constructed during 1988.

The spillway excavation and dam construction will be completed during 1989 and 1990. The instrumentation contract, which will include all of the instruments that can be installed from the surface of the completed dam and from ground surface, will be awarded during 1990. The dam contract will be substantially completed by December 1990. Construction of two of the road embankments and the permanent road surfacing will be deferred until 1991 to avoid interference with the work of the spillway contractor.

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1.0 INTRODUCTION

1.1 Project Description

The Oldman River Dam project will provide flow regulation and onstream storage of a dependable supply of water which will permit multi-purpose use and irrigation service area expansion in Southern Alberta. The project will be constructed at a site located a short distance downstream of the confluence of the Oldman River, Crowsnest River and Castle River. At this location the Oldman River has eroded a steep sided, narrow valley through bedrock and makes a southerly turn before resuming its northeasterly course through Lethbridge to eventually join the Bow River and form the South Saskatchewan River.

After construction the dam will occupy most of that part of the river valley that is orientated in the north-south direction. The spillway will be positioned on the right bank with the chute orientated in the direction of the river downstream of the project site. Both the spillway and the dam have been located to minimize the volume of excavation and fill required, and to maximize the use of soil and rock available from the required structure excavations in the dam fill.

During construction, the Oldman River will be diverted through two circular concrete lined tunnels 6.5 m in diameter and each approximately 900 m long. The tunnels are located through the left bank and will have a diversion capacity of 1000 m^3/s. After completing their diversion function, the tunnels will be converted to low level outlets for riparian and multi-purpose releases. Control of reservoir outflow will be by means of a 2 m diameter hollow cone valve and two small riparian valves installed in a concrete plug at the upstream third point of each tunnel. The maximum discharge capacity of the low level outlets will be 100 m^3/s through each tunnel.

The 76m high dam will be a zoned earth and rock fill embankment with a central core of clay till. The shells of the dam will be constructed of soil and rock obtained from the required spillway excavation and sand and gravel taken from the river alluvium deposits in the vicinity of the project site. Internal filters and drains of sand and gravel will control seepage through the dam and its foundation.

The spillway will be 110 m wide and will have seven, 10 m wide gated openings. The gates will be 9.6 m high. The spillway will pass the 1:1000 year flood of 2970 m^3/s at full supply level. During the routed probable maximum flood of 9320 m^3/s the reservoir will be surcharged 5.9 m above full supply level and the spillway will then discharge 7200 m^3/s. The flood discharges will be conducted back to the river through a converging chute terminating in a 40 m wide flip bucket energy dissipator.

Closure of the reservoir will be completed by a 1.6 km long earth fill dyke of 10 m maximum height.

1.2 Purpose of the Report

This report has been prepared to present the concepts, criteria, parameters and methods used to design the following major project structures and features:

- the earth and rock fill dam across the river valley,

- the earth fill dyke located across the low saddle to the northeast of the dam,

- the excavation of the spillway approach channel and for the spillway headworks, chute, flip bucket and plunge pool,

- the drainage tunnels in both river banks, which will also provide access to the reservoir control valve chamber,

- the grout curtain from west of the spillway, across the river valley along the dam axis and to the east of the valve chamber,

- five road embankments,

- parts of the permanent road across the top of the dam, dyke and spillway approaches and

- instrumentation to measure the performance of the dam.

This report supplements the Preliminary Engineering Report (UMA/Acres (1986)) and essentially supersedes the design brief for these structures, which was first issued in September 1986. The design brief was prepared during the early stages of final design to outline the design concepts and criteria that would be used. It was updated periodically to reflect changes in the design details that resulted during the design process, and as a result of various studies and the additional field and laboratory data that became available during final design period. The design will be constantly reviewed during construction and will be modified where necessary.

1.3 Contract Packaging

The project structures described in this report will be constructed under eight contracts. These are:

- Dam Contract.
- Dyke Contract.
- Drainage Tunnels Contract.
- Road Embankment No. 1 Contract.
- Road Embankment Nos. 2 and 3 Contract.
- Riprap Supply Contract
- Road Surfacing Contract.
- Instrumentation Contract.

For simplicity this design report has not been subdivided by contract. The report covers the design of the major works included in the eight contracts.

The other two major project contracts are the Diversion Tunnels Contract and the Spillway Contract. The spillway excavation and construction of road embankment Nos 4 and 5 are part of the Dam Contract.

1.4 Structure Locations

The general location of the dam, dyke, drainage tunnels and the five road embankments with respect to the other project structures is shown on Plates 1 and 2.

The structures were located following an optimization study which considered the technical and economic advantages of different arrangements of the dam, spillway and diversion tunnels.

The location of the main body of the dam is predicated on the need to provide minimum clearance at the downstream toe from the edge of the plunge pool that will develop in time and to minimize the length of the diversion tunnels. Within these constraints the dam axis has been orientated in a straight line from the high ground on the left bank across the valley. At the right side of the valley the dam axis curves to the south through a gully to meet the river bank on the south slope of the gully approximately at right angles to the contours. This alignment will necessitate shaping of the core contract area within the narrow steep sided gully.

On the left bank the dam alignment follows the high ground along a 500 m long freeboard section orientated to the northeast.

The dyke has been located across a low saddle to the northeast of the freeboard section of the dam and on the upstream side of the TransAlta Utilities Corporation 69 kV powerline that crosses the site. The toe of the dyke will be approximately 5 m from the powerline right-of-way at its closest point. In this location the development of Borrow Area "A" as a source of dam and dyke construction materials can be extended as far downstream as possible without incurring the cost of relocating the powerline.

The drainage tunnels will be downstream of, and approximately parallel to, the grout curtain. The drainage tunnel and grout curtain on the right bank will pass beneath the spillway headworks and will extend eastwards through the right abutment of the dam. The left drainage tunnel will pass through the left abutment of the dam, downstream of the grout curtain, and will connect to the valve chamber area of the diversion tunnels. In this area the drainage tunnel will be enlarged to form the valve equipment gallery. The left drainage tunnel will extend further into the left river bank to the east of the valve equipment gallery.

The spillway orientation aligns the discharge in the direction of the Oldman River valley downstream with the plunge pool at its closest location to the right river bank. The approach channel has been located to minimize the volume of excavation without introducing undesirable hydraulic conditions in the curved entrance to the spillway headworks.

Road embankment Nos. 1, 2 and 3 will be located on the main access road to the project site from the south. Road embankment Nos. 4 and 5 will provide access from the left abutment of the dam to the permanent access road from the construction camp area and to the permanent road to the right end of the dyke respectively.

1.5 Reservoir Levels

The following reservoir levels have been established for design of the major structures:

Full Supply Level (FSL)	1118.6 m

This will be the maximum operating level of the reservoir for all river flow conditions up to and including the 1:1000 year flood.

Maximum Reservoir Level	1124.5 m

The reservoir will be allowed to surcharge above FSL during floods exceeding the 1:1000 year event up to the maximum reservoir level during the Probable Maximum Flood (PMF)

Minimum Operating Reservoir Level	1064.0 m

The reservoir will provide 490,000 dam^3 of live storage over its operating range.

2.0 GEOLOGY

2.1 Site Investigation

The field investigation and laboratory testing program was initiated in May 1985 and was essentially completed by September 1987. The program comprised two basic components; the investigation of the foundation conditions at the project site and the search for naturally occurring construction materials. The extent of the field work and laboratory testing carried out are summarized in Tables I and II. The location of test holes are shown on plates in Appendix B. Full details of the site investigation are given in the Preliminary Engineering Report (UMA/Acres (1986)), the 1986 Site Investigation Data Report (UMA/Acres (1987)), and the 1987 Site Investigation Data Report (UMA/Acres (1988)).

TABLE I

FIELD PROGRAM SUMMARY

	Foundation Investigation		Construction Materials	
	(No)	(m)	(No)	(m)
Vertical cored holes (rock)	82	3000	-	-
Inclined cored holes (rock)	4	350	-	-
Rotary holes (overburden)	27	330	6	240
Becker hammer holes (overburden)	46	290	12	120
Auger holes (overburden)	55	610	25	620
Exploratory shafts	4	128	-	-
Test pits trenches, and surface exposures	93	-	237	-

TABLE II

LABORATORY TESTING SUMMARY

TEST	FOUNDATION MATERIALS	CONSTRUCTION MATERIALS
Natural Water Content	420	1104
Visual Classification	-	663
Atterberg Limits	470	156
Gradation Analysis	444	483
X-ray Diffraction	20	-
Slaking	31	-
Swelling Pressure	6	-
Water Soluble Sulphates	43	4
Consolidation	7	10
Unconfined Compression	129	8
Triaxial Compression	33	53
Direct Shear (Peak and Residual)	233	-
Dynamic Modulous and Poisson's Ratio	9	-
Point Load Tests	38	-
Cation Exchange Capacity	-	5
Organic Content	-	4
Dispersion	-	6
Permeability	-	8
Standard Proctor Compaction	-	33
Maximum and Minimum Density	-	7
Los Angeles Abrasion	-	13
Magnesium Sulphate	-	13

2.2 Foundation Conditions

The regional setting for the site geology is fully described in the Preliminary Engineering Report (UMA/Acres (1986) and in Mollard (1985)).

2.2.1 Overburden

The overburden soils on the right bank consist of, from top down, topsoil, eolian silt, glaciolacustrine silt and clay, sand, gravel and clay till to a maximum depth of 16 m. The overburden soils on the left bank consist of topsoil, eolian silt, glaciofluvial sand and gravel, glaciolacustrine silt and clay and clay till to a maximum thickness of 30 m beneath the dyke. There is a buried channel passing to the north of the project site and veering to the southeast of the dam and dyke. Bedrock surface drops from the vicinity of Borrow Area "A" towards the northeast resulting in substantial overburden thickness.

In the valley bottom, the foundation soil is mainly pervious river alluvium consisting of well to poorly graded sandy gravel with traces of silt and clay overlain by approximately 1 m of a relatively loose alluvial silty sand in the centre of the valley increasing to 5 to 6 m near the right bank. The total depth of alluvium ranges from 2 to 12 m. Permeability of the alluvium is estimated to range between 1×10^{-2} and 35 cm/s based on grain size data.

Several metres of colluvium overlie, and interfinger with, the river alluvium in many areas of the dam foundation particularly upstream of the dam axis on the right bank. The colluvium consists of silty clayey sand and clayey silt with Standard Penetration Test "N" values ranging between 4 and 15 blows per 300 mm.

The river banks feature many areas of exposed weathered bedrock outcrop.

On the left bank, there is an area of pervious sandy deposits which probably will extend into the foundation of the freeboard section of the dam and the right abutment of the dyke.

2.2.2 Bedrock

Rocks underlying the abutments and foundations of the dam are part of the Porcupine Hills and Willow Creek formations. The Porcupine Hills formation overlies the Willow Creek formation and the contact is located at approximate elevation 1006 m at the dam axis. Both formations consist of interbedded sequences of freshwater sandstone and mudrock (mudrock is defined as rock consisting of siltstone and claystone). It is believed the rocks were deposited in a meandering fluvial system where sandy deposition was restricted to major channels. Deposition of fines occurred on levees and in flood basins.

Two major stratigraphic units within the Porcupine Hills Formation and one in the Willow Creek Formation have been identified during exploration at the project site. These have been designated in ascending order; the Lower Mudrock, which is the uppermost part of the Willow Creek Formation and the Basal Sandstone and the Upper Mudrock sequences, which are at the bottom of the Porcupine Hills Formation. At the dam axis the contact between the Basal Sandstone and the Upper Mudrock is at elevation 1040 approximately. A full description of the regional and site geology is included in Appendix B.

2.2.3 Bedrock Shear Strength

A detailed assessment was made of the shear strength of the bedrock in the foundation of the dam, in the slopes of the spillway excavation and in the river banks upstream and downstream of the dam. The bedrock strength that will control stability is a combination of the strength across the beds and along the subhorizontal structural discontinuities or seams found to exist within the rock mass. The foundation investigations have concentrated on obtaining data relative to these seams. The strength, continuity, thickness, roughness and orientation of the seams were analyzed using data from coring, drilling, laboratory testing and detailed mapping of the geology revealed in the test tunnel, test shafts, test trenches and surface exposures and the rock exposed in the portal excavations for the diversion tunnels and the construction access road on the left river bank.

The lack of any natural bedrock instabilities in the site area, and the construction performance of the surface and underground excavations confirms that present design assumptions are conservative.

The strength of individual seams in situ will be less than that of the adjacent intact rock due to the deformation that has taken place, but will be somewhat greater than the minimum residual value measured in laboratory direct shear tests on samples taken from the seam (Skempton (1964), Patton (1961), Nicholson (1983)). A design strength representative for each seam has been assessed by considering:

a) its representative residual shear strength which recognizes the variability of the laboratory test data (Skempton (1964), USCE (1970)) and,

b) its "geological component" which recognizes the variability of the characteristics of the seams across the site and of the site geology (Barton and Choubey (1977), Barton (1973), Skempton and Petley (1967), Hamel and Spencer (1984), Imrie and Taylor (1985)).

The represenative residual shear strength is a shear strength value chosen from the distribution of values characteristic of a given seam across the site. Its selection tends to the conservative (i.e. low) side of the mean residual shear strength values measured in the laboratory (USCE (1970)). The values measured are consistent with the plasticity, clay content and mineralogy of the mudrock (Skempton (1964), Townsend and Gilbert (1974)). The geological component takes into account the actual conditions in and along the seam, including the thickness, roughness and shape of the seam, and that portion of peak shear strength that still exists where the deformation that caused the seam was insufficient to reduce the shear strength completely to its residual value.

All of the seams that have been identified in Appendix B do not have the same influence on the stability of the dam and excavation and existing slopes. Most seams can be disregarded due to the lack of continuity in the vicinity of the dam or slope (Imrie and Meidal (1985)). The shear strength of thirteen major seams that have the most significant effect on the design of the dam and of the excavated and existing slopes are listed in Table III.

TABLE III

DESIGN SEAMS

SEAM NO.	REFERENCE ELEVATION(1) (m)	DESIGN EFFECTIVE ANGLE OF SHEARING RESISTANCE	STRUCTURE (2)
V12	1030.7	12.5°	Dam
V16(3)	1046.8	17.0°	Dam
V18	1049.4	16.0°	River Bank
R1	1068.2	11.5°	)
R6	1084.5	13.5°	)
R7	1086.5	13.0°	)River Bank
R8	1087.5	12.5°	)Stability
R9	1088.5	11.5°	)and
R13	1096.5	15.5°	)Spillway
R14	1099.8	16°	)Excavation
R16	1105	13°	)
R17	1107.7	16.5°	)
R18	1115	12.5°	)

NOTES:

1) Elevation of seam at the dam axis for Seams V12, V16 and V18 and elevation of seam at the spillway centreline for the remaining seams
2) This column indicates the part of the project that is most affected by the seam.
3) Details of the derivation of this table are given in Appendix B.

Seam V12 is the uppermost seam in the mudrock sequence found within the Basal Sandstone. It has the lowest shear strength of all seams in this mudrock sequence. It is probably only continuous under the upstream portion of the dam but for the design it has been assumed to be continuous through the entire dam foundation.

Similarly seam V16 has been assumed to be continuous, although drill hole and other data suggest that it probably is not present in 70 percent of the dam foundation.

Seam V18 is above the top of bedrock in the river valley and only has an affect on the stability of the dam abutments and river banks.

There is strong evidence that all of the other seams listed in Table III are continuous through the river banks and they have been considered in the stability analyses of the river bank and spillway excavation slopes.

2.2.4 Permeability

Based on water pressure testing the permeability of the rock above elevation 1070 m in the river banks is generally high, ranging up to 8×10^{-3} cm/s. Between elevations 1070 m and 1040 m the permeability values are significantly less than those above elevation 1070 m. The values range from less than 1×10^{-5} cm/s to 4×10^{-4} cm/s. The left bank is apparently more pervious than the right bank.

Permeability values in the Basal Sandstone in the abutments range from less than 1×10^{-5} cm/s to 2×10^{-4} cm/s. Permeabilities below elevation 1040 m in the valley bottom, however are generally higher, ranging up to 6 x 10^{-3} cm/s. This increase in permeability, although not reflected in the joint frequency observed in the drill core, is consistent with the valley rebound phenomena. Below elevation 1010 m permeabilities vary from less than 1 x 10^{-5} cm/s to 5 x 10^{-4} cm/s.

A very pervious zone was encountered in the two shafts located in the river bottom. It appears to be associated with Structural Unit No.1 around elevation 1030 m. Seepage inflow into one of the shafts was so great that it was impossible to enter the shaft for logging and core sampling. An unsuccessful attempt was made to dewater this shaft using dewatering wells.

Existing groundwater levels in the river banks are generally close to river level near the valley and rise within the range of elevation 1050 m to 1060 m in the dam abutment and spillway areas. The permeability of the glaciolacustrine silt and clay and clay till is generally low.

2.3 Construction Materials

To the maximum extent possible the dam and dyke will be constructed from soil and rock material from the spillway and other required excavations. This will be supplemented by material from borrow sources close to the project site. An inventory of suitable naturally occurring material has been made and the most suitable deposits have been allocated to the various contracts. The dam core will be constructed of clay till from the spillway excavation and from Borrow Area "A". Processed granular materials for the filters, drains and riprap bedding and pitrun pervious material for the upstream shell of the dam will be obtained from recent alluvial and terrace deposits in the Oldman River valley and from abandoned upland terraces adjacent to the river valley. These sources are all within 6 km upstream and 3 km downstream of the dam. Some excavated rock from the required excavations of the diversion tunnels inlet and outlet portals and all of the excavated rock from the spillway approach channel, headworks, chute, flip bucket and plunge pool will be placed in the dam. The dyke will be constructed of the glaciolacustrine clay that must be excavated from Borrow Area "A" to expose the clay till for use in the dam core. Riprap for upstream slope and downstream toe protection will be obtained from a limestone quarry in Crowsnest Pass selected by the Riprap Supply contractor. The site of the quarry is approximately 75 km to the west of the project site.

Full details of the field and laboratory work to locate and test construction materials are given in Appendix A.

3.0 DAM

3.1 General

The main section of the dam that will be located across the river valley is described in this section. The freeboard section of the dam that extends across the uplands on the left river bank is described together with the dyke as both structures have the same design requirements, the same foundation conditions and the same earth fill cross section.

The right abutment of the dam will end 160 m from the left face of the spillway. On the left bank the freeboard portion of the dam will run northeast following high ground to the 1125.5 m contour some 600 m from the top of the river bank.

3.2 Dam Section

The 76 m high dam will be a zoned earth and rock fill structure. The dam section will be essentially the same as that described in the Preliminary Engineering Report and is shown on Plate 3. The central core will be wide, with its width at any point approximately equal to the head. It will be constructed of clay till with shells of sandstone, siltstone, claystone, soil from the required excavations, and sand and gravel from the river valley alluvium. Approximately 180,000 m^3 of rock excavated from the diversion tunnels inlet portal will be placed in the upstream part of the dam by the Diversion Tunnels contractor. The core will be protected on both sides by filter zones and will be drained on the downstream side by a chimney zone that will lead to a drainage blanket beneath the downstream shell of the dam. These filter and drain zones will be constructed of processed granular materials from river alluvium and terrace deposits. A thick layer of free-draining pitrun sand and gravel from the same sources will be placed in the upstream shell of the dam to improve stability during reservoir drawdown. The upstream slopes and downstream toe will be protected from erosion by a layer of quarried rock riprap on a bedding zone of coarse processed cobbles and gravel. The crest road will be an asphalt surfaced public road.

A positive cutoff to bedrock will be achieved by excavating and backfilling a core trench to relatively impervious bedrock. A two, and possibly three, line grout curtain will be constructed beneath the dam core trench across the river valley and into the dam abutments.

Soil and rock fill cofferdams including grout curtains installed beneath the cofferdams will be used to divert and control the river and to permit dewatering of the core trench and construction of the lower part of the dam.

The downstream cofferdam will be located separate from, and 300 m downstream of, the dam, to permit dewatering of the spillway flip bucket and plunge pool excavation as well as the dam foundation. The main access across the river to the spillway area will pass over the top of the downstream cofferdam. The downstream cofferdam will be removed prior to the project being placed in service.

The dam will be constructed in stages, over three construction seasons, while the river is diverted through the diversion tunnels.

3.3 Dam Foundation

3.3.1 Foundation Excavation

The core trench in the bottom of the river valley will be excavated to the Basal Sandstone. The overburden and loose, weathered bedrock will be removed within the core contact area on the river bank slopes. Rock surfaces on the river banks will be presheared to specified lines to produce a smooth surface against which the core and adjacent filter and drain zones will be placed. The objective will be to expose sound bedrock to slopes which will not be steeper than 1 horizontal to 1 vertical. Some shaping of the core contact area will be necessary at the base of the right bank where the dam axis follows an eroded gully. Here the bedrock surface is adversely sloped and would otherwise create undesirable deformation of the bottom of the core. Beneath the dam shells, all potentially liquefiable sandy alluvium and colluvium material in the river bottom will be removed. In the remainder of the river valley, minimal excavation will be required to expose competant sand and gravel alluvium. On the river banks the colluvium will be removed to expose weathered bedrock in the dam foundation.

Soil foundations on the uplands will be flattened to a stable slope. These excavated slopes will be extended upstream and downstream above and beyond the dam slopes. Overburden excavation on the uplands will be limited to the removal of topsoil, stripping to a minimum of 500 mm below original ground surface, and the excavation of a 2 m minimum depth core trench with a 5 m base width.

3.3.2 Cofferdam Cutoff

It will be necessary to dewater the construction area in the river valley to excavate and prepare the dam foundation to excavate the core trench and to place fill in the lower part of the dam. Dewatering will be achieved behind cofferdams constructed upstream and downstream of the dam. The upstream cofferdam will be incorporated into the toe of the dam.

The upstream cofferdam will be constructed to elevation 1060 m. Further protection upstream will be provided by placing a temporary impervious layer to elevation 1080 m on the upstream face of the dam. This temporary impervious layer will be connected to the core of the upstream cofferdam and will provide protection against overtopping during flood flows up to the 1:50 year event. This layer will be removed and replaced with riprap slope protection before reservoir impounding. To be effective both cofferdams will require a cutoff through the pervious water bearing alluvial sand and gravels to the underlying bedrock in the river bottom.

The upstream cofferdam will be constructed in two stages. Each stage will have a cutoff to rock. The cutoff beneath the first stage or starter cofferdam will be a trench, excavated to bedrock below the water table, into which Zone 1A Impervious Fill material will be progressively dumped and allowed to slide to the bottom. The cutoff will be only partially effective, but will provide a sufficient barrier during low river flows to permit a second trench located just downstream of the first to be excavated and backfilled in the dry with compacted Zone 1 Core material. This will form the cutoff beneath the second stage cofferdam. Plate 4 illustrates the cofferdams and their cutoffs.

The contract specifications allow the use of a slurry trench as a contractor's designed alternative to the fill cutoff. If a slurry trench is used beneath the starter cofferdam, the second cutoff will not be required.

The downstream cofferdam will have either an open trench replacement or a slurry trench cutoff.

To reduce seepage into the dewatered dam foundation and core trench and plunge pool excavations, the foundation of both the second stage upstream cofferdam and the downstream cofferdam will be grouted to cut off the pervious zone in the Basal Sandstone that was detected during the investigation of the dam foundation at a depth of around 20 m below original surface in the river valley.

The dam foundation will be maintained dry by pumping behind the cofferdams until the fill is brought up to cofferdam level.

3.3.3 Dewatering

Control of seepage inflows will be required during core trench excavation and during treatment of the core contact area. Upward seepage through vertical joints in the Basal Sandstone is a possibility during construction.

200 mm diameter tube wells will be installed on both sides of the core trench in the river bottom and around the spillway flip bucket and plunge pool excavation as part of the specified dewatering requirements. The tube wells will

extend down through the alluvial sand and gravels and into the pervious zone in the Basal Sandstone to elevation 1035m. Initially the tube wells will be spaced at 20m centres and then will be split spaced if necessary. Pressure relief drains will also be specified to handle seepage flows from the floor of the core trench. The contractor will be required to make his own assessment of the existing conditions and to develop a dewatering scheme which is required by the contract specifications to include the grout curtain beneath the cofferdams and tube wells around the excavations.

3.3.4 Dam Cutoff

Seepage paths close to ground surface beneath the dam will be intersected by the core trench which will be located beneath the core of the dam and backfilled with compacted core material.

On the uplands, the core trench will extend a minimum depth of 2 m below stripped foundation level through the frost lossened surficial zone to an impervious layer. The core trench will be extended downwards to cut-off any pervious sandy soils that will be encountered in the left bank and that are found to be continuous beneath the dam. The pervious soils will be located by augering holes along the bottom of the core trench. The core trench will be continued from the east end of the freeboard section of the dam to connect to the cutoff trench of the dyke at its west abutment.

3.3.5 Bedrock Surface Treatment

The required excavated slopes of the bottom of the core trench generally follow the slopes of the existing river valley. Steeper slopes on the river banks will be accepted locally to a maximum of 1 horizontal to 1 vertical. Blasting of the rock will not be permitted except as part of the preshearing and postshearing of the core trench in the dam abutments and to flatten, and make safe, the ledge sandstone that exists on the upper part of the left abutment. In the river bottom, blasting will not be permitted and ripping of the mudrock to be removed will be required. The shaped excavated surface on the river banks will be cleaned with air jets, the surface cracks will be slush grouted. The entire surface will be covered with a layer of shotcrete to protect the excavated mudrock surface for one or two years until fill placement and to provide a working surface for the grouting operation. The use of low pressure water monitors will only be permitted to clean the Basal Sandstone in the core trench in the valley bottom. The surface of the sandstone rock of the core trench in the valley bottom will be slush grouted and shaped with dental concrete, but will not be covered with shotcrete.

The shotcrete protection on the river banks will be removed prior to fill placement, only where it has separated from the excavated rock surface and can easily be removed. The rock surface exposed will then be cleaned with air jets, and slaked and otherwise deteriorated rock, will be removed. The resulting surface will be re-shaped and backfilled with concrete. The shotcrete will not be replaced.

Seepage of water from the slopes will be intercepted with piped drains and will be conveyed to sumps at the bottom of the slope. Seepage into the core trench in the valley bottom will be conveyed through ditches to sumps from which the water will be pumped.

The core contact area will be consolidation grouted using low pressures through inclined holes to a depth of 5 m and over a 5 m by 5 m grid pattern. If grout takes are high, a second grid of 5 m deep holes will split space the primary grid.

After foundation preparation, the portion of the core trench that has not been covered to a depth greater than 3 m with Zone 1 Core material prior to winter shut down, will be protected against frost action with 3 m layer of uncompacted unclassified fill or other insulation during the winter shut down period. No traffic will be allowed to pass over this fill. The fill will be removed and the surface of the prepared foundation will be cleaned with air jets prior to commencing the placement of the Zone 1 Core material. Where Zone 1 Core material has been placed, the surface of the fill will be scarified and if necessary moisture conditioned before fill placement recommences.

3.3.6 Curtain Grouting

The exploration of the dam and spillway foundations which included pressure packer testing, has identified the need for, and the feasibility of, grouting the foundation bedrock. The grout curtain will be a conventional installation (Casagrande (1961), Houlsby (1982), Deer (1982) and Albritton (1982)), similar to those used at most dam sites to reduce groundwater seepage through joints, cracks, bedding planes and other structural discontinuties that exist in all bedrock (for example, Dickson Dam, W.A.C. Bennett Dam (Jory and Imrie (1968)), and Mica Dam (Pratt, McMordie and Dundas (1972))). The rock beneath and around the impounding structure will be injected with cement grout, under pressure through drillholes. As well as reducing the loss of water from the reservoir the grout curtain together with the foundation drainage system installed from the drainage tunnels will reduce groundwater pressures beneath the structures and in the rock and soil slopes downstream of the dam.

The grout curtain which is shown on Plate 5, will be about 1,000 m long and will extend in the abutments from the bedrock surface down to the approximate top of the Basal Sandstone at elevation 1035 m and in the valley bottom from bedrock surface down to the top of the Lower Mudrock Sequence at elevation 1000 m. Beneath the dam abutments, the grout curtain base will slope upward from elevation 1000 m to 1035 m at a low angle of about 20° to ensure adequate grouting of the potentially pervious river banks.

On the left abutment, the grout curtain will tie into the ring grout curtain to be constructed around the diversion tunnels and just upstream of the valve chamber area as part of the Diversion Tunnels contract. On the right abutment, the grout curtain will extend to the west of the spillway headblock.

A two line grout curtain will be used and will extend across the river bottom and into both abutments. The need for, and depth of, a third line between the other two lines will be decided during construction on the basis of the grout takes of the previous two lines and from pressure packer test results in exploratory drill holes to be located at 24 m spacing along the third grout line. All grout holes will be orientated 30° to the vertical along the dam axis to maximize the intersection of the holes with horizontal and vertical joint sets.

The grout holes will also be orientated 10° to the vertical in an upstream direction to intersect the orthogonal subvertical joint sets. The grout curtain will be positioned approximately at the upstream third point of the core contact area, as shown on Plate 5.

Grouting will be done from the bottom of the drillhole upwards using a single packer set at intervals to be set during construction. It is anticipated that the interval will be 3 m on average.

Each line of the grout curtain will comprise primary and secondary grout holes at 6 m and 3 m spacing respectively. If grout takes are high, tertiary holes will be grouted at split spacing of 1.5 m. The depth of the tertiary grout holes will be varied to target zones of high grout take. A grout take of more than 60 kg/m of drillhole or a water take corresponding to a coefficient of permeability greater than 1×10^{-5} cm/s will be the criteria for establishing the need for additional grouting.

The maximum allowable grouting pressure at the packer will be related to the effective weight of overlying rock. In shallow bedrock, the grouting pressures will not exceed the effective overburden weight. At depth, grouting pressures will slightly exceed the effective overburden weight. Adjacent to the river valley slopes, the grouting pressures will be reduced to suit the lower lateral confining pressures insitu.

Cement grout with bentonite will be specified. An initial water-cement ratio of 3:1 by mass will be specified and adjusted according to the rate of grout take. Some of the grout holes will be cored and pressure packer tested to test the effectiveness of grouting.

Grouting will be scheduled to be done after all blasting in the vicinity has been completed and before the pressure relief drains have been drilled and installed.

3.3.7 Pressure Relief

A positive cutoff to bedrock and the grout curtain will reduce seepage past the dam. The presence of the alluvial gravels beneath the dam shells will control exit gradients of any seepage under the dam.

Pressure relief drains will be used to improve the stability of the abutment portions of the dam and the downstream river bank slopes. Drainage tunnels will be driven in each abutment parallel to, and generally about 30 m downstream of, the grout curtain. 75 mm diameter pressure relief drain holes inclined at 30^{o} to the vertical will be drilled upward and downward along the tunnels at a spacing of 2.5 m. These drain holes will extend to within 3 m of bedrock surface and into the Basal Sandstone. Short pressure relief drain holes will be located at 45^{o} to the vertical to relieve pressures behind the shotcrete liner of the drainage tunnels. The pressure relief drain holes will be lined with a tightly fitting double slotted polyvinyl chloride pipe system with a removable filter cloth sleeve to prevent migration of particles clogging the drains. A fan of pressure relief drain holes will radiate from the four ends of the drainage tunnels.

3.4 Fill Materials

The dam section and the zones of the different materials from which the dam will be constructed are shown on Plate 3. The specified gradations of the zones are shown on Plates 6 and 7.

The central core, Zone 1, will be constructed of clay till. The horizontal width of the core at bedrock foundation, and at any section along the dam axis, will not be less than the head of water on the upstream side of the core at that section. The core will be protected on the downstream side with a sand filter material placed in Zone 2. A more pervious processed gravel drain will be placed in Zone 3 downstream of Zone 2. Both of these zones will be continued beneath the downstream shell to form a drainage blanket that will intercept foundation seepage and prevent the migration of the fine grained material from the dam and the dam foundation into the drains. The drainage blanket will exit at the toe of the dam. On the upstream side, the core will be protected with a layer of Zone 2 Filter material.

The dam shells will utilize, in Zones 4 and 4A, all suitable soil and rock from the required spillway excavation and some of the rockfill from the diversion tunnels portal excavation. Only rock from the required excavations will be accepted in Zone 4. The excavated rock will be mainly mudrock with a minor content of sandstone. Zone 4A is a weight zone in which glaciolacustrine clay, sand, gravel, cobbles, clay, till and rock from the required excavations will be placed. Zone 4A has been located in the centre of the downstream shell where shear strength is not critical. The extent of Zones 4 and 4A has been established on the basis of stability and drainage requirements and an analysis of the availability of the different materials from the required excavations. During fill placement oversize rock pieces will be routed to the narrow Zone 4B on the outer face of the downstream shell.

Selected pitrun sand and gravel will be placed in Zone 5 in the upstream shell to enhance drainage and stability during reservoir drawdown. It was determined during a full scale crushing, washing, screening, classifying and blending test that a surplus of gravel size material will be obtained during the production of Zone 2 Filter, Zone 3 Drain and Zone 7 Riprap Bedding materials. This surplus gravel product will also be placed in Zone 5 on the outer part of the upstream shell beneath the riprap bedding.

Zone 2 Filter material will also be placed in between the shell and Zone 3 Drain in the downstream shell to prevent the fine grained material that will come from the claystone component of Zones 4 and 4A after it has slaked, from migrating into and clogging the Zone 3 Drain material.

Zone 8 Riprap material placed on the Zone 7 Riprap Bedding layer will protect the upstream face of the dam from the erosive action of waves. Zone 8 Riprap will also be placed on the downstream toe of the dam to protect it against high velocity back eddies during the operation of the flip bucket.

Note that the following sections, that describe the various embankment fill materials and their specifications, have been written to cover their use in the dyke and road embankments and on the protected river banks as well as the dam. This avoids excessive repetition in later sections.

3.4.1 Zone 1 - Core

Clay till will be used for the impervious zones of the dam and cofferdam. It will be obtained from the required spillway excavation and the Borrow Area "A" located on the left bank upstream of the dyke as shown on Plate 1. The surficial glaciolacustrine silt and clay must be excavated from the spillway excavation and the borrow area before the underlying clay till can be removed. The clay till consists of an upper layer of high plasticity clay till underlain by medium plasticity silty clay till.

The laboratory test data on the clay till are listed in Appendix A. A statistical analysis of the in situ moisture contents is presented on Plates A-5, A-7 and A-8 in Appendix A.

Zone 1 Core material will be well graded within the envelope shown on Plate 6. Minimal processing will be required to remove the low cobble and boulder content. The maximum particle and lump size will be restricted to 100 mm maximum. The placement moisture content will be specified to be between one percent greater than, and one percent less than Standard Proctor optimum moisture content. This will achieve maximum deformability of the core with minimal cracking, consistent with adequate strength for trafficability and overall stability. Zone 1 Core material will be placed in uncompacted layers not thicker than 200 mm and will be compacted with a 50 ton pneumatic tired roller.

Zone 1 Core material will be placed at a higher moisture content of between one and three percent above Standard Proctor optimum moisture content and in thinner lifts of 100 mm uncompacted thickness in two regions of the dam core. These are a 2 m thick zone measured perpendicular to prepared foundation at the bottom of the core trench and between elevations 1110 m and 1122 m across the full width of the top of the dam core. The latter will provide a more plastic zone in the core where deformation and the potential for the development of cracks is critical.

The core of the upstream cofferdam will also be constructed of Zone 1 Core material unless a slurry trench is used.

The specifications and construction details for the Dam contract are summarized in Table IV and are based on the results of the a test fill built of clay till from an excavation adjacent to Borrow Area "A" as part of the Access Roads Contract.

TABLE IV
ZONE 1 CORE
SPECIFICATIONS SUMMARY

LOCATION	-	Dam core
	-	Upstream cofferdam core
VOLUME REQUIRED	-	1,500,000 m^3
SOURCES	-	Spillway Excavation
	-	Borrow Area "A"
ACCESS	-	Across tunnel inlet portal
	-	Axis road
MINING	-	Glaciolacustrine silt and clay to be removed first to road fills, dyke or stockpile
	-	Borrow Area "A" excavation to be limited upstream of dyke.
	-	Some stockpiling of spillway excavation will be necessary so that the headworks area of the spillway excavation can be handed over to the Spillway contractor before fill can be placed in the core trench.
PROCESSING	-	Till to be broken down to 100 mm maximum size lumps.
	-	Moisture to be added in the borrow pit or stockpile area only.
	-	Only supplemental water may be added at the fill to prevent drying.
	-	Placement moisture content to be +1% to -1% of optimum moisture content.
	-	Placement moisture content to be +1% to +3% of optimum moisture content within 2 m of the core contact surface in the valley bottom and at the top of the core.
	-	Oversize particles and oversize till lumps may be removed on the fill.
	-	Large sand and gravel lenses found in till will be spoiled.
PLACING	-	200 mm maximum uncompacted layer.
	-	100 mm maximum uncompacted layers within the +1% to + 3% moisture content zone.
	-	100 mm layers to be sloped upwards towards valley wall core contact surface.

TABLE IV (Continued)

	- Surface of each compacted layer 70 mm minimum depth to be scarified before placing next layer. - Moisture conditioning will not be permitted on the fill, except for the application of water to maintain the specified moisture content.
COMPACTION	- 50 t (or 100 t) pneumatic tired roller. - 6 coverages minimum (to break down lumps as well as to compact). (One coverage is the number of passes necessary to cover the entire surface with the compactor including overlap). - Compacted density to be equal to or greater than 100% Standard Proctor maximum density.
MEASUREMENT AND PAYMENT	- Unit price per m^3 of fill in the dam measured from the drawings. - Unit prices per m^3 of fill measured in stockpiles for Zone 1 from the spillway excavation that cannot be placed directly in the dam. - Unit price per m^3 of fill measured in the spoil disposal area for stripping topsoil from spillway excavation, and Borrow Area "A" from which Zone 1 material is obtained. - Unit price per m^3 of fill measured in the spoil disposal area for spoiling glaciolacustrine silt and clay that must be removed to obtain Zone 1 material from Borrow Area "A" but is in excess of total Zone 1A requirements. - Unit price per m^3 for excavation from spillway excavation, Borrow Area "A" and the required stockpiles. - No separate payment for haul. - No allowance will be made for settlement that occurs during construction.

3.4.2 Zone 1A - Impervious Fill

The freeboard section of the dam, the dyke and the temporary blanket on the upstream face of the dam will be constructed of Zone 1A Impervious Fill material which will be specified to include glaciolacustrine clay, clay till and any other suitable fine grained soils obtained from the required excavations, Borrow Area "A" or any other source. Zone 1A Impervious Fill material will also be used in the upstream starter cofferdam to provide initial cut-off through the river alluvium and the five road embankments. Data on the glaciolacustrine deposits are given in Appendix A.

A statistical analysis of the in situ moisture contents is presented in Plates A-6 and A-9 in Appendix A.

Zone 1A Impervious Fill material will be well graded within the envelope shown on Plate 6. Minimal processing will be required to remove the low cobble and boulder content. The maximum particle and lump size will be restricted to 100 mm maximum. The placement moisture content will be specified to be between one percent greater than, and one percent less than, Standard Proctor optimum moisture content. Zone 1A Impervious Fill material will be placed in uncompacted layers not thicker than 200 mm and will be compacted with a 50 ton pneumatic tired roller.

The specifications and construction details are summarized in Table V and are based on the results of the a test fill built of glaciolacustrine clay obtained from an excavation adjacent to Borrow Area "A" as part of the Access Road Contract.

TABLE V
ZONE 1A IMPERVIOUS FILL
SPECIFICATIONS SUMMARY

LOCATION	- Dam Freeboard Section - Dyke - Road embankment Nos 1, 2, 3, 4 and 5 - Starter cofferdam of dam - Temporary upstream blanket on the dam - Upstream and downstream cofferdams
VOLUME REQUIRED	- 450,000 m^3
SOURCES	- Glaciolacustrine clay (or till) from Borrow Area "A". - Glaciolacustrine clay (or till) from the spillway excavation.
ACCESS	- Road to dyke. - Across tunnel inlet portal.
MINING	- Topsoil, organic and other unsuitable material will be removed to spoil disposal area. - Break down to maximum 100 mm lumps.
PROCESSING	- Stockpiling required for road only. All other stockpiling to be contractor's choice. - Moisture to be added in the borrow pit or stockpile area only. - Only supplemental water may be added on the fill to prevent drying. - Placement moisture content to be +1% to -1% of optimum moisture content (dam seal, dyke and road embankments).
PLACING	- 200 mm maximum compacted layers (dam seal, dyke and road embankments). - Scarify surface of each compacted layer 70 mm minimum depth before placing next layer (dam seal, dyke and road embankments). - Moisture conditioning not permitted on fill, except for the application of water to maintain the specified moisture content.
COMPACTION	- Dam seal, dyke and road embankments only. Other fills may be compacted at contractor's option. - 50 t (or 100 t) pneumatic tired roller.

TABLE V (Continued)

	- 6 coverages minimum (to break down lumps as well as to compact). - Compacted density to be equal to or greater than 100% Standard Proctor maximum density.
MEASUREMENT AND PAYMENT	- Unit price per m^3 of fill in the dam, dyke, road embankment Nos 1, 2 and 3, starter cofferdam and temporary upstream blanket measured from the drawings. - Unit price per m^3 of fill measured in the spoil disposal areas for stripping dam, dyke and road embankments foundations and borrow areas from which Zone 1A materials is obtained. - No extra payment for stockpiling. - Unit price per m^3 for spillway, dam, dyke and road embankment foundation excavation. - Lump sum price for downstream cofferdam and adjacent road construction. - No separate payment for haul.

3.4.3 Zone 2 - Filter

A processed granular filter zone is required between the core, the downstream drainage zone and the upstream shell of the dam, to blanket the bedrock abutments downstream of the core and between the foundation and the downstream drainage blanket. Considering the fine gradation of the core, its potential for cracking, and the very coarse nature of the alluvium shell material, close control of the gradation of this material is required. The gradation of this material has been selected to meet the filter design criteria relative to adjacent materials. Permeability tests will be carried out during construction to check that the permeability after placement and compaction is greater than 1×10^{-2} cm/s. Zone thicknesses of 1 m for horizontal zones and 3 m for steeply inclined zones have been specified. Minimal compaction adjacent to the core has been specified. The gradation limits are shown on Plate 6. A uniformity coefficient between 4.0 and 6.0 has been specified. Gap graded material will not be accepted. Naturally occurring granular materials available in the river alluvial and terrace borrow areas will not meet the required gradation limits without crushing, screening, washing, classifying and blending.

Zone 2 will be placed in 250 mm uncompacted layers and will be compacted lightly with a vibrating smooth steel-drum roller.

The specifications and construction details specified are summarized in Table VI.

TABLE VI
ZONE 2 FILTER
SPECIFICATIONS SUMMARY

LOCATION	- Downstream and upstream of dam core - Downstream of dam drainage blanket - Dyke blanket drain - Cofferdam toe drain - Cofferdam core filters - Dyke drainage blanket
VOLUME REQUIRED	- 550,000 m^3
SOURCES	- Borrow Areas B1, B2, B4, B6, B7, B10, B11, B12, B13, B14 and B15. - Dam core trench excavation. - Private off-site sources will be allowed.
ACCESS	- Contractor to design and construct a road to upstream borrow areas. - Project access road to other borrow areas.
MINING	- Topsoil, organic and other unsuitable material to be removed to worked out parts of the borrow pits. - Excavation below river level will be required for Borrow Areas B1, B2, and B10 to B14 inclusive.

TABLE VI (Continued)

PROCESSING	- Crushing, screening, washing, classifying and blending will be necessary.
PLACING	- 250 mm maximum uncompacted layers. - 1 m thick horizontal zones. - 3 m wide for steeply inclined zones. - Surface of Zone 2 to be at least 150 mm above all adjacent zones except Zone 3 at all times. - Construction traffic will not be allowed to travel along the zone except at a limited number of locations. - Planked protection of the zone where construction traffic needs to cross the zone.
COMPACTION	- Smooth steel-drum vibrating roller. - One coverage only for zone adjacent to core with no compacted density requirements. - 2 coverages to a minimum compacted density between 95% and 98% of Standard Proctor maximum density for horizontal zones.
MEASUREMENT AND PAYMENT	- Unit price per m^3 of fill in the dam and dyke measured from the drawings. - Unit price per m^3 of fill measured in the spoil disposal areas for stripping topsoil and overlying unsuitable material from borrow areas from which Zone 2 is obtained. - Unit price per t.km for hauling to the dam and dyke from processing area. - Unit price per t. (same as haul mass) for borrow area excavation of Zone 2. - No separate payment for material excavated and processed but not placed in the dam or dyke.

3.4.4 Zone 3 - Drain

A processed granular drain zone is required downstream of the downstream filter zone and as the downstream drainage blanket. The gradation limits of Zone 3 material will meet the filter design criteria relative to the gradation of the adjacent materials. Permeability tests will be carried out during construction to check that the permeability after placement and compaction is greater than 1 cm/s. Zone thicknesses of 2 m for horizontal zones and 3 m for inclined zones have been specified. Minimal compaction in the inclined portion of the zone has been specified. The gradation limits are shown on Plate 6. A uniformity coefficient between 3.0 and 6.0 has been specified. Gap graded material will not be accepted. Crushing, screening, washing, classifying and blending will be required to meet the specified gradation.

Zone 3 Drain will be placed in 250 mm uncompacted layers and will be compacted with a vibrating smooth steel drum roller.

The specifications and construction details are summarized in Table VII.

TABLE VII
ZONE 3 DRAIN
SPECIFICATIONS SUMMARY

LOCATION	- Downstream of dam core - Downstream of dam shell blanket
VOLUME REQUIRED	- 320,000 m^3
SOURCES	- Borrow Area B1, B2, B4, B6, B7, B10, B11, B12, B13, B14, B15. - Dam core trench excavation. - Private off-site sources will be allowed.
ACCESS	- Contractor to design and construct a road to upstream borrow areas. - Project access road to other borrow areas.
MINING	- Topsoil, organic and other unsuitable material to be removed to worked out parts of the borrow pits. - Excavation below river level will be required for Borrow Areas B1, B2, and B10 to B14 inclusive.
PROCESSING	- Crushing, screening, washing, classifying and blending will be necessary.

TABLE V11 (Continued)

PLACING	- 250 mm maximum uncompacted layers. - 2 m thick horizontal zones. - 3 m width for steeply inclined zones. - Surface of Zone 3 to be between 150 mm and 300 mm above the adjacent Zone 2 at all times. - Construction traffic will not be allowed to travel along the zone except at a limited number of locations. - Planked protection of the zone where construction traffic needs to cross the zone.
COMPACTION	- Smooth steel-drum vibrating roller. - One coverage only and no compacted density requirements for zones adjacent to core. - Two coverages to a compacted density between 95% and 98% of Standard Proctor maximum density for horizontal zones.
MEASUREMENT AND PAYMENT	- Unit price per m^3 of fill in the dam measured from the drawings. - Unit price per m^3 of fill measured in the spoil disposal areas for stripping topsoil and overlying unsuitable material from borrow areas from which Zone 3 is obtained. - Unit price per t.km for hauling to the dam from processing areas. - Unit price per t. (same as haul mass) for borrow area excavation of Zone 3. - No separate payment for material excavated and processed but not placed in the dam.

3.4.5 Zones 4 and 4B - Rock Fill and Zone 4A - Unclassified Fill

All of the downstream shell, and approximately one half of the upstream shell will be constructed of material from the required excavations or from sources of Zone 1A and Zone 5 material. The main source of Zone 4 and 4A material will be the excavation for the spillway approach channel, headworks, chute, flip bucket and plunge pool. Zone 4 and 4A materials will also be obtained from the diversion tunnel portal excavations that will have been either placed in the dam or stockpiled as part of the Diversion Tunnels contract.

Zone 4 Rock Fill material may include claystone, siltstone and sandstone, oversize rock particles from glacial till, river alluvium and terrace deposits. Zone 4 Rock Fill material has a 300 mm maximum size limitation. The dam section includes a zone of limited size at the downstream edge that will accept sandstone and siltstone between 300 mm and 1 m size to minimize the constraints imposed by this requirement on the contractor's excavation methods and to produce a downstream face that will not weather and become unsightly with time. This oversize zone is called Zone 4B. With the exception of the need to drill and blast the spillway excavation to produce rockfill meeting the specified size limitations and to separate out any oversize material no processing of Zone 4 material will be required.

Zone 4A Unclassified Fill may include excavated rock, glacial till, glaciolacustrine clay, outwash sand and gravel, river alluvium and terrace deposits with minimal moisture content limitations.

Zone 4 Rock Fill material will be placed in 500 mm uncompacted layers. Each material component of Zone 4A Unclassified Fill material will be placed in the uncompacted layers specified for that material. Zones 4 and 4A material will be selectively placed so that the fill will be relatively homogenous with no significant voids left between large size rock particles. Zone 4 Rock Fill material and Zone 4A Unclassified Fill material will be spread with a bulldozer blade and compacted with vibrating rollers, or pneumatic tired rollers, depending on the material.

The specifications and construction details are summarized in Tables VIII and IX and are based in part on the rock test fill constructed of excavated rock from the tunnel inlet portal as part of the Diversion Tunnels contract. The timing of this test fill was such that the excavated rock slaked and otherwise weathered in a stockpile for about four months before being placed and compacted in the dam. The test fill results were therefore not representative of rockfill placed directly into the dam immediately after it has been excavated.

TABLE VIII
ZONE 4 and 4B ROCK FILL
SPECIFICATIONS SUMMARY

LOCATION	- Dam shells
VOLUME REQUIRED	- 1,900,000 m^3
SOURCES	- Sandstone and mudrock from spillway excavation. - Sandstone and mudrock stockpiles left by the Diversion Tunnels contractor.
ACCESS	- To be designed and constructed by the Contractor.
MINING	- Blasted or ripped excavation to achieve 300 mm maximum particle size requirement with some sandstone and siltstone to 1 m maximum size. - Stockpiling of the rock excavated from the spillway headworks and chute will be necessary.
PROCESSING	- No processing other than removal of particles larger than 300 mm size to Zone 4B.
PLACING	- 500 mm maximum uncompacted layers. - Moisture will be added prior to arrival and on the fill. - Rock particles between 300 mm and 1 m will be moved to Zone 4B in the dam section.
COMPACTION	- Systematic routing of hauling and placing equipment.

Table V111 (Continued)

- Smooth steel-drum vibrating roller for the sandstone and siltstone component and 50 t (or 100 t) pneumatic tired roller for the claystone and siltstone component.
- 8 coverages of the appropriate roller.

MEASUREMENT AND PAYMENT - Unit price per m^3 of fill in the dam measured from the drawings.
- No separate payment for haul.
- Unit price per m^3 for spillway excavation.

TABLE IX
ZONE 4A UNCLASSIFIED FILL
SPECIFICATIONS SUMMARY

LOCATION - Dam shells

VOLUME REQUIRED - 1,100,000 m^3

SOURCES - Zones 1A, 4 and 5 sources.

ACCESS, MINING, PROCESSING, PLACING, COMPACTION - All as for Zone 1A, 4 and 5.

MEASUREMENT AND PAYMENT - Unit price per m^3 of fill in the dam measured from the drawings.
- Separate payment for haul per t.km for Zone 5 material only.
- Unit price per m^3 for spillway excavation and Borrow Area "A" excavation.
- Unit price per t. for Zone 5 material excavation only.

3.4.6 Zone 5 - Granular Fill and Zone 5A Road Sub-base

The outer portion of the upstream shell of the dam, the cofferdams and other minor zones will be constructed of Zone 5 Granular Fill material obtained from river alluvial deposits. Zone 5A Road Sub-Base material will come from the same source and will be placed in the sub-base of the permanent access road, located on top of the dam and dyke, on the downstream face of the dam, across the downstream toe of the dam to the drainage tunnels and on other maintenance access roads. The specified gradation for Zone 5 Granular Fill material shown on Plate 6 is broad enough to cover all the sand and gravel found in the river alluvial deposits without processing other than the removal of occasional oversize particles. Selective loading will be required in the borrow pit to avoid including more than five percent of the particles from the overlying silty alluvium finer than the #200 sieve size. Oversize materials larger than 500 mm size will be moved on the fill to Zone 4 or Zone 7. Some of the material is gap-graded and placement methods will be specified to avoid segregation.

Zone 5 will be placed in 600 mm thick uncompacted layers and compacted with vibratory smooth steel drum rollers.

Zone 5A Road Sub-Base will have the same specified gradation limits as Zone 5 but will be compacted to a density not less than 98 percent of Standard Proctor maximum density.

The limits and construction details to be specified are summarized in Table X.

TABLE X

ZONE 5 GRANULAR FILL AND ZONE 5A ROAD SUB-BASE SPECIFICATIONS SUMMARY

LOCATION	- Pervious upstream dam shell. - Cofferdam shells. - Transition zone downstream of dam core and blanket drain. - Beneath the riprap bedding layer on the dam and the dyke. - Dyke toe drain. - Road sub-base on the dam and dyke.
VOLUME REQUIRED	- 2,500,000 m^3
SOURCES	- Borrow Areas B1, B2, B4, B6, B7, B10, B11, B12, B13, B14 and B15. - Dam core trench excavation. - Stockpiles left by Diversion Tunnels contractor.
ACCESS	- Existing left bank construction access roads and access road along the river valley.
MINING	- Topsoil, organic, fine grained alluvium and colluvium and other unsuitable non-pervious materials will be removed and placed either in Zone 1A or in worked out parts of the borrow pits. - Excavation mostly under water working first one side of the river then the other will be required for Borrow Areas B1, B2, and B10 to B14 inclusive. - Stockpiling of dam core trench excavation may be necessary.
PROCESSING	- No processing other than spoiling fine grained alluvium and colluvium such that not more than 5% is finer than the #200 sieve size and removal of oversize particles.
PLACING	- 600 mm maximum uncompacted layers. - Particles larger than 500 mm size will be moved on the fill to Zone 4 or Zone 7. - Segregation of uniformly graded material will not be permitted. - Smooth steel-drum vibrating roller. - 4 coverages for Zone 5. - Systematic routing of hauling and placing equipment. - 98% of Standard Proctor maximum density for Zone 5A.

TABLE X (Continued)

MEASUREMENT AND PAYMENT	- Unit price per m^3 of fill in the dam and dyke measured from the drawings. - Unit price per m^3 of fill measured in the spoil disposal areas for stripping topsoil and overlying unsuitable material from the dam core contact area, and borrow areas from which Zone 5 and Zone 5A are obtained. - Unit price per t.km for hauling to the dam and from each of the borrow areas. - Unit price per t. (same as haul mass) for borrow area excavation for Zone 5 and Zone 5A. - No separate payment for material excavated but not placed in dam, dyke or road embankments.

3.4.7 Zone 6 - Permanent Road Gravel and Zone 6A Temporary Road Gravel

The permanent road across the Oldman River will pass across the top of the dam, dyke road embankments and spillway and will connect to the access road to the construction camp and project bridge. In the vicinity of the spillway and the dam the road will have an asphalt surface. Along the dyke and the road embankments the surface will be gravelled. The roads will be completed as part of the Road Surfacing contract in 1991 after the completion of the dam and dyke construction. A temporary road gravel, Zone 6A of 75 mm minimum thickness will be placed as part of the Dam and Dyke Contracts. Permanent road gravel will be placed on the freeboard section of the dam and on permanent maintenance roads.

The specifications and construction details of the Zone 6 Permanent Road Gravel and Zone 6A Temporary Road Gravel to be specified are summarized in Table XI.

TABLE XI

ZONE 6 PERMANENT ROAD GRAVEL AND ZONE 6A TEMPORARY ROAD GRAVEL
SPECIFICATIONS SUMMARY

LOCATION	- Crest of dam and dyke. - Permanent MD road from north side of road embankment No. 1 to spillway and between spillway and dam. - Permanent maintenance roads.
VOLUME REQUIRED	- 50,000 m^3
SOURCES	- Zone 2, 3 and 5 sources.
ACCESS	- Same as Zone 2, 3 and 5.
MINING	- Same as Zone 2, 3 and 5.
PROCESSING	- Same as Zone 2, 3 and 5 to produce Alberta Transportation Designation 2, Class 40 aggregate for Zone 6 and Designation 4 Class 20 for Zone 6A.
PLACING	- Single layers. - Moisture conditioning on the fill.
COMPACTION	- Compacted density will be greater than 98% Standard Proctor maximum density.
MEASUREMENT AND PAYMENT	- Unit price per m^3 of fill in the dam, dyke and on the road sections measured from the drawings. - Unit price per t.km for hauling to the dam from each of the borrow areas. - Unit price per t. (same as haul mass) for borrow area excavation.

3.4.8 Zone 7 - Riprap Bedding

Bedding materials are required below the riprap to form a transition between the riprap and the underlying finer sand and gravel on the upstream face of the dam, dyke and the protected areas of the river bank. Zone 7 Riprap Bedding material for the dyke will be obtained by the Riprap Supply contractor from the rock spalls by-product that passes through the grizzly used to screen the riprap into its various gradations. This riprap bedding will be placed in stockpiles at the site. Zone 7 Riprap Bedding material for the dam and the river bank protection will be obtained from the river alluvial deposits as part of the granular processing operation to produce Zone 2 Filter and Zone 3 Drain materials. No compaction will be required. The required gradation for Zone 7 Riprap Bedding material meets the filter design criteria with respect to Zone 3 Drain and Zone 5 Granular Fill material on which it is placed. The gradation is shown on Plate 7.

The specifications and construction details to be specified are summarized in Table XII.

TABLE XII
ZONE 7 RIPRAP BEDDING
SPECIFICATIONS SUMMARY

LOCATION	- Beneath riprap on the upstream face of the dam and dyke, the lower downstream face of the dam, and on the protected areas of the river banks.
VOLUME REQUIRED	- 150,000 m^3
SOURCE	- Borrow Areas B1, B2, B4, B6, B7, B10, B4, B12, B13, B14, and B15 (Dam Contract). - Stockpile by Riprap Supply Contractor (Dyke Contract).
PROCESSING	- Screening, washing and blending will be necessary. - By-product of the riprap quarry operation for Zone 8A, 8B, 8C, 8D.
PLACING	- Placed and spread on the completed face of the dam and dyke, and on the filter layers on the protected river banks. - May not be end dumped and pushed down the face of the dam, dyke or the protected river banks. - Top of Zone 7 to be not more than 2 m above any of the Zones 8 at any time.
COMPACTION	- No compaction other than systematic routing of the hauling and placing equipment.
MEASUREMENT AND PAYMENT	- Unit price per t. measured by weigh scale.

3.4.9 Zone 8 - Riprap

Upstream slope protection is required on the face of the dam and dyke and on some of the river bank slopes to prevent damage from wave action. Scour protection is also required at the downstream toe of the dam to protect against strong eddy currents produced by spillway discharges.

Riprap, meeting the durability and other specified property requirements listed in Table XIII, will be obtained and delivered to the project site under the riprap supply contracts. The riprap will be screened and sorted before delivery and will be placed at the site into stockpiles and will be placed as part of the Dam and Dyke contracts. A guaranteed delivery schedule will be included in the Dam and the Dyke contracts.

TABLE XIII
RIPRAP PROPERTY REQUIREMENTS

TEST	METHOD	REQUIREMENTS
Abrasion Resistance	ASTM Designation C535 (Los Angeles Test)	Less than 40 percent loss of weight after 500 revolutions.
Freeze-Thaw	AASHTO T103 (Full Immersion Ledge Rock Gradation	Less than fifteen percent of the particles split after 20 cycles
Sodium Sulphate Soundness	ASTM Designation Designation C88 (Magnesium Sulphate, Coarse Gradation	Less than 10 percent loss of weight after 5 cycles
Specific Gravity	ASTM Designation C127	2.60 minimum
Absorption	ASTM Designation C127	2 percent maximum

Eight sources of hard durable rock suitable for riprap have been investigated. Of these the limestone from Summit Lime Works, (a commercial quarry in the Crowsnest Pass), a volcanic breccia prospect at Iron Ridge, near Coleman and limestone from the Frank Slide within the right-of-way reserved for twinning Highway No. 3 and on the north side of Frank Lake were considered to be able to provide rock of suitable size, gradation and quality. The other sources were local sandstones that will not meet the durability criteria. None of the bedrock available at site is suitable for use as riprap for the permanent structures as it does not meet all of the durability and other criteria listed in Table XIII. The use of the Frank slide material would require the approval of Alberta Culture, Alberta Environment and Alberta Transportation. The actual source was specified to be selected by the riprap supply contractors.

The design and selection of the various riprap gradations and that of the riprap bedding are described in Section 8.6. The gradations are shown on Plate 7.

A fifth class of riprap, Zone 8E material was specified to be used for temporary slope protection during the construction period. Zone 8E will be used on the cofferdams. The specification will allow the use of sandstone from the channel deposits found at the site particularly on the upper part of the left river bank. The Diversion Tunnels contractor will stockpile some of this riprap at the Diversion Tunnels inlet portal. The specified gradation limits for Zone 8E will be a maximum size of 1 m, 50 percent larger than the 150 mm size and 20 percent larger than the 100 mm size.

The specifications and construction details of Zones 8A, 8B, 8C, and 8D only are summarized in Table XIV.

TABLE XIV
ZONES 8A, 8B, 8C, AND 8D RIPRAP
SPECIFICATIONS SUMMARY

LOCATION	- Upstream face of the dam, dyke and adjacent river valley walls. - Downstream face of the dam.
VOLUME REQUIRED	- 280,000 m^3. (Total of all four classes of riprap).
SOURCES	- Stockpile at project site by riprap supply contractors whose sources will be selected by the contractors:
ACCESS (SUPPLY CONTRACT)	- Highway No. 3 and project access road.
MINING (SUPPLY CONTRACT)	- Drill and blast quarry operation. - Rehabilitation will be required.
PROCESSING (SUPPLY CONTRACT)	- Rough grizzly separation of quarry product by the riprap supply contractors. - Stockpiling will be necessary. - No segregation permitted in stockpiles. - Final sorting, blending and placing by the Dam and Dyke contractors. - Riprap will be placed in a single lift thickness in the stockpile.
PLACING	- Riprap will be placed directly on the slope and not dumped at the top of the fill and pushed down the slope. - Larger particles will be uniformly distributed and smaller particles will fill voids to form an interlocked mass, through which Zone 7 cannot be seen.
COMPACTING	- No compaction
MEASUREMENT AND PAYMENT	- Unit price per t. measured as delivered to, and placed in stockpiles at the project site by the riprap supply contractors. - Unit price per t. measured as delivered to, and placed in the dam and dyke and on the excavated slopes from the riprap stockpiles.

4.0 DYKE AND FREEBOARD SECTION OF DAM

4.1 General

The reservoir requires the construction of a dyke approximately 1.6 km long to contain the reservoir on the left bank. Approximately 1.1 km of the dyke will be freeboard dyke constructed above full supply level. The maximum height of the dyke will be 10 m, with the average height being 3.5 m. The dyke cross section is shown on Plate 8. The dam cross section will transition to a similar cross section for a distance of approximately 500 m on the left bank where its foundation is above full supply level. This length of the dam has been called the "freeboard section of the dam" and is also covered in this section as it has similar design requirements and a similar section.

4.2 Foundation Conditions

The soil stratigraphy on the left bank consists of a shallow mantle of topsoil underlain by glaciolacustrine silt, clay, and fine silty sand and clay till. The glaciolacustrine sand deposits are stratified and are locally pervious. The thickness of the overburden varies from 20 m at the top of the left river bank to 30 m at the east end of the dyke. Bedrock of the Upper Mudrock underlies the overburden.

4.3 Foundation Treatment

The dyke will be founded on the overburden after the topsoil and organic matter have been removed to a minimum of 500 mm below original ground surface. To reduce seepage through frost loosened soil and any pervious sandy layers that may be found to be continuous in the dyke and freeboard dam foundations, a cut-off trench of minimum 2 m depth and 5 m base width will be excavated to an impervious layer.

Auger holes will be located along the cutoff trench to establish that pervious layers near the surface in the foundation will be cut off. If pervious layers are encountered at the base of the trench, it will be deepened locally to cutoff that zone. The cut-off trench will be backfilled using compacted Zone 1A Impervious Fill material. Pervious soils exposed by excavation of the left dam abutment and in Borrow Area "A" will be blanketed with Zone 1 Core material if they can form a leakage path.

No other measures are considered necessary for control of seepage through the dyke foundation.

4.4 Dyke and Freeboard Dam Zones

The main body of the dyke and freeboard dam will be constructed from Zone 1A Impervious Fill which will be predominantly the glaciolacustrine clay that must be removed from the Borrow Area "A" to expose the clay till that will be used to construct the dam core. The dyke and freeboard dam will have upstream and downstream slopes of 4 horizontal to 1 vertical to accommodate the variable and low strength glaciolacustrine clay material. Zone 1A Impervious Fill material is described in Section 3.4.2.

A drainage blanket of Zone 2 Filter material will be constructed where the stripped foundation downstream is at or below elevation 1119.0 m to provide seepage control for the dyke and freeboard dam.

The downstream slopes of the dyke and freeboard dam will be covered with a layer of topsoil and will be seeded.

The upstream slopes will be protected from wave action by Zone 8 Riprap material placed on Zone 7 Riprap Bedding as described in Sections 3.4.8 and 3.4.9. A filter of Zone 3 Drain and then Zone 2 Filter materials are required beneath the Zone 7 Riprap Bedding to protect the underlying Zone 1A Impervious Fill material.

5.0 SPILLWAY EXCAVATION

The excavation for the spillway approach channel, headworks, chute and flip bucket and plunge pool is shown in Plate 2. Typical sections are shown on Plate 9. The soil and rock conditions that will be encountered in the excavations are described in Section 2.2. 700,000 m^3 of glaciolacustrine clay from the spillway excavation will be used to construct the three road embankments to the south of the spillway. The remaining 300,000 m^3 will be used in Zone 4A in the downstream shell of the dam and in the construction of access roads, cofferdams and other temporary facilities. 650,000 m^3 of the underlying clay till will be placed in the lower part of the dam core. An estimated 3,000,000 m^3 of bedrock excavation will be required to complete the spillway excavation. All of the rock excavated will be placed in the dam. Construction scheduling and contractual interfacing between the Dam and Spillway contracts will necessitate some stockpiling and double handling of the excavated material before it is placed in the dam or road embankments. The excavation for the spillway headworks must be completed first to allow the Spillway contractor to start construction of the concrete works. Stockpiling of some of the Zone 1 Core material will be necessary as the excavation and foundation preparation of the core trench and the grouting will not have been completed before the headworks excavation must be handed over to the Spillway contractor. Similarly Zone 1A Impervious Fill material for two of the three road embankments to the south of the spillway will have to be stockpiled as these embankments cannot be constructed until after the Spillway contract has been completed.

The base of the spillway excavation by the Dam contractor will be completed to within 2.5 to 3 m of the final foundation lines of the concrete structures. Adjacent slopes will be excavated to the final lines. The remainder of the excavation, which will be to complex surfaces required to suit the concrete headworks, chute, flip bucket, associated retaining walls and other works, will be completed by the Spillway contractor. The shape of the bottom of the excavation to be completed by the Dam contractor has been simplified. The thickness of rock left to be excavated is based on an analysis of the penetration of the frost front during the coldest winter on record.

The excavated slopes of one horizontal to one vertical with 10 m wide benches shown on Plate 9, will be stable against deep seated failure without support as described in detail in Section 8.1.3. Surface sloughing may occur with time. The contract will allow for local rock bolting and dowelling and for applying shotcrete to locally stabilize the excavated slopes. The excavated soil slopes adjacent to, and downstream of, the spillway headworks will be covered with topsoil and seeded.

6.0 DRAINAGE TUNNELS

This section describes the design of the drainage tunnels. It does not cover the design of the pressure relief drains that will be drilled from the drainage tunnels. These are covered in Section 3.3.7.

Drainage tunnels will be constructed in both river banks and underneath the spillway headworks to control groundwater seepage downstream of the grout curtain and to reduce pore water pressures under the spillway headworks and in the excavated and existing slopes (USCE (1982), Sherard (1963)).

The left bank drainage tunnel will also serve as the access to the valve equipment gallery in the diversion tunnels. The valve equipment gallery will be an enlargement above the drainage tunnels.

The layout for the drainage tunnels is shown on Plates 1 and 5. The location of the drainage tunnels was governed by the following considerations.

- location of core contact area of the dam in the abutments and in the dyke areas
- grout curtain location
- spillway headworks location
- access to the portals
- tailwater levels
- bedrock stratigraphy
- location of the diversion tunnels and valve chamber areas

6.1 Design Parameters

The design parameters, listed in Table XV are representative of the materials that will be encountered along the length of the drainage tunnels and were used in the design.

TABLE XV

DESIGN PARAMETERS

	MUDROCK	SANDSTONE
Uniaxial Compressive Strength	0.8 - 50 MPa	20 - 110 MPa
Modulus of Elasticity (E)	400 - 1500 MPa	2 - 20 GPa
Poissons Ratio	0.4 - 0.5	0.3 - 0.4
Horizontal to Vertical Stress Ratio k_o		
- near portals	0.5	0.5
- in tunnels	1.0	1.0
Unit Weight	23.6 kN/m^3	23.6 kN/m^3
Joint Spacing	0.3 - 1 m	0.3 - 1 m

The joint spacing will not significantly affect on the design of the 2.8 m diameter tunnels.

No major seams exist in the rock through which the drainage tunnels will be driven, as shown in Table III. Other seams in the horizontally bedded rock will have negligible influences on the stability of the tunnels and the portals beyond minor local instability that will be handled with rock bolts and shotcrete.

6.2 Portals

The portals were designed for at least 3 m of clear rock cover above the tunnel crown. The portal face will be near vertical and will be supported with rock bolts and shotcrete. A cast-in-place concrete structure will be used to transition from the face to the entrance through a section backfilled with granular fill.

6.3 Tunnels

Either drill and blast methods or mechanical excavators such as tunnel boring machines or road headers will be acceptable for construction.

The alignment of the drainage tunnels was selected to be close to the grout curtain without a high hydraulic gradient developing along flow lines between the grout curtain and the drains in the tunnels. The alignment is controlled by the location of the valve chamber and of the spillway. In general the drainage tunnels were set as low as possible to produce a maximum reduction of porewater pressures developing within the river banks due to the presence of the reservoir. The main purpose of the drainage tunnels is to stabilize the excavated and existing slopes of the area.

The invert levels of the tunnel portals were controlled by the 1:100 year flood tailwater level at elevation 1055.2 m. A gradient of 0.2 to 0.5 percent was used to create a positive drainage towards the portals. A sump and buried drain discharging below minimum tailwater level will ensure year-round drainage and a dry entrance at the portal.

6.4 Tunnel Cross-Section

Since the drainage tunnels do not need a concrete lining the cross-section of the tunnel used was an optimum shape for minimizing bending stresses in the shotcrete lining. It was estimated that a maximum of 2 m of rock load will be exerted at the crown of the tunnel which is equivalent to a load of

50 kPa. This load may differ in the field depending on the nature of the rock. Spot bolting and thicker shotcrete will be used when such fractured areas are exposed during tunnelling. The tunnel cross section is shown on Plate 10.

6.5 Roof Support

From the available data from the drillhole cores, shafts and in particular from the full scale experience of the behaviour of the rock in the test tunnel, the rock masses along the drainage tunnel location were classified using CSIR Geomechanics classification as proposed by Bieniawski [Hoek and Brown (1980)].

Based on a Rock Mass Rating of 60-41, Class III, the following rock support design was adopted. The support design was compared with the methods proposed by Steiner and Einstein (1980).

a) For running tunnel

- Pattern roof dowels in rows of 3 dowels at 1.25 m centre to centre spacing along tunnel with one vertical and one at 45^{0} to the vertical on each side. The rock dowels will be resin grouted epoxy coated 25 M reinforcing steel bars 1.6 m long.
- 75 mm shotcrete; consisting of 50 mm fibre reinforced and 25 mm plain on the inside surface as protection against injury from protruding fibres.

b) For enlargements and tunnel adjacent to the portals for a distance of 10 m from the portal face.

- Pattern roof dowels in rows of 3 dowels at 1.25 m centre to centre spacing with one vertical and one at 45° to the vertical on each side. The rock dowels will be resin grouted epoxy coated 25 M reinforcing steel bars 1.6 m long.

- 150 mm shotcrete; consisting of 125 mm fibre reinforced and 25 mm plain.

c) In all tunnels a shorter horizontal dowel will be located on each side at the spring line at 2.5 m centre spacing. These rock dowels will be resin grouted epoxy coated 25 M reinforcing steel bars.

6.6 Tunnelling Advance

During design the permissible stand-up time for unsupported rock excavation was examined and the following specification was developed:

"Tunnel Advance Limits

The following excavation advance limits are maxima, and may be reduced as dictated by ground conditions encountered. The exposed surfaces shall be sealed to prevent surface deterioration within these limits.

a) From the portal face for first ten metres of the Drainage Tunnels:

   - Advance length 1.5 m maximum, specified support to be completely installed to the face within 8 hours of the ground being opened.

b) From ten metres beyond the portal face for the next ten metres of the Drainage Tunnels and in the Valve Equipment Gallery:

   - Advance length 3 m maximum, specified support to be completely installed to the face within 8 hours of the ground being opened.

c) Remainder of the Drainage Tunnels:

- Advance 5 m maximum, support to be completely installed to the face within 16 hours of the ground being opened."

7.0 RIVER DIVERSION AND CONSTRUCTION STAGING

7.1 River Diversion Scheme

River diversion will be performed in two stages. The diversion scheme and construction sequence are shown schematically on Plates 11 to 13 inclusive.

7.1.1 Stage I

The Diversion Tunnels contractor will excavate the tunnel portals and drive the diversion tunnels behind cofferdams protecting both the inlet and outlet portals. Diversion channels excavated in the river deposits around the upstream portal cofferdam and downstream of the diversion tunnel outlet works will be required. This system will be used until after the 1988 spring flood. Both upstream and downstream cofferdams will be left for the Gates and Stoplogs contractor to dry test the stoplogs and will then be removed by the Dam contractor.

7.1.2 Stage II

The Dam contractor will divert the river through the two 6.5 m diameter concrete lined tunnels on the left bank by initially cofferdamming the river with a low cofferdam and then constructing the upstream shell of the dam to act as the cofferdam.

7.2 Hydraulic Considerations

The Stage I diversion works have been designed to pass a 1:50 year flood without overtopping. The diversion channel has been sized to pass the flow with an upstream waterelevation at 1057.1 m. The diversion tunnel inlet cofferdam will be built to elevation 1058.2 m allowing 1.1 m of freeboard above the 1:50 year flood level.

During Stage II the diversion tunnels will be required to pass all river flows following the Spring flood of 1988 until the Fall of 1990 when plugging of the tunnels and installation of the control valves will commence and the reservoir will be allowed to fill. This period will include two peak river flow seasons.

River closure and initial diversion will be accomplished following the Spring flood of 1988 by constructing the upstream cofferdam to elevation 1060 m. The temporary impervious blanket will be placed on the upstream face of the dam to elevation 1080 m before the Spring flood of 1989 in order to pass floods less than or equal to the routed 1:50 year flood of 1045 m^3/s through the diversion tunnels with a freeboard of 1.4 m and a downstream water level of 1053.1 m. During 1989, the upstream shell of the dam will be raised to above elevation 1091 m which will then provide protection during the 1990 season for floods less than or equal to the routed 1:300 (approximate) year flood of 1300 m^3/s with a freeboard of 1 m and downstream water level of 1054 m. If the upstream shell is completed to elevation 1100 m the flood protection will be for the routed 1:1000 year event of 1420 m^3/s.

The downstream cofferdam located downstream of the spillway flip bucket and plunge pool excavation will be constructed to a crest elevation of 1054 m which will provide about 1 m freeboard above the tailwater level during the 1:50 year flood.

The use of a 1:50 year flood for the design of the cofferdams and diversion works involves a risk of overtopping and the potential for damage to the permanent works and downstream property. The risk involved is low compared with current river diversion practice on projects of this magnitude, which are often designed for more frequent floods.

7.3 Cofferdams

7.3.1 Upstream Cofferdam

The upstream cofferdam and temporary upstream blanket that will protect the fill placement work will be constructed in the following stages. Plate 4 shows the upstream cofferdam section. The upstream impervious blanket is shown on Plate 3.

First, random fill will be dumped across the river connecting to the portion of the diversion tunnels inlet cofferdam, that does not need to be removed to permit the river flows to enter the tunnels. The starter cofferdam will then be constructed to elevation 1054 m by dumping Zone 1A materials from the required excavations into a trench excavated through the river valley alluvium as described in Section 3.3.2. This is shown on Plate 11. With the river flows diverted and river cutoff across most of the valley alluvium , the full cutoff will be constructed on the downstream side of the starter cofferdam. The excavated sand and gravel alluvium will be placed upstream and downstream of the Zone 1 Core to form the cofferdam to above elevation 1060 m as shown on Plate 12.

Following completion of this cofferdam to elevation 1060 m the temporary upstream blanket of Zone 1A Impervious Fill material will be placed on the upstream face of the dam to elevation 1080 m. This blanket will be connected to the core of the upstream cofferdam as shown on Plate 3.

Placement of the upstream shell of the dam will continue to at least elevation 1091 m as shown on Plate 14 during the 1989 construction season. The upstream blanket will be removed down to elevation 1060 m after the core has been constructed above elevation 1080 m, and will be replaced with riprap slope protection.

7.3.2 Downstream Cofferdam

Only one downstream cofferdam will be used for both the dam and spillway. It will be located as shown on Plate 1. It will be constructed with a cross-section similar to the upstream cofferdam with shells of tunnel spoil and river alluvium, a compacted core of Zone 1A Impervious Fill material and a Zone 1A Impervious Fill core trench or slurry trench cutoff. The cofferdam will be removed by the Dam contractor at the end of construction.

7.4 Construction Staging

The construction staging for the project shown schematically on Plates 11 to 14 inclusive was developed during design to establish feasibility, to determine the start and completion dates for each contract and to identify the inter-contract relationships. The overall construction schedule and the target and mandatory completion dates for the various stages have been selected considering the most cost effective use of construction equipment and such design constraints as pore pressure dissipation in the core, and the sequential staging of blasting, grouting and then pressure relief drain installation. Key dates for completion of certain portions of the work, that relate to interfacing with other contractors, will be specified in each contract.

The following items relate to the staging envisaged during design. The actual construction sequence will differ as the contractors will be allowed to the maximum extent feasible to develop plans that best satisfy their own particular equipment and resources.

a) The Dyke, Road Embankment No. 1 and Drainage Tunnels contracts will be awarded during the Fall of 1987. The Dam contract will be awarded in Spring 1988.

b) The spillway excavation will be started by the Spillway Access Road contractor who will excavate some 250,000 m^3 of glaciolacustrine silt and clay to construct the spillway access road. Ten rebound points will be installed under this contract. This work will be completed by September 1987.

c) Road Embankment No. 1 contract will include excavation of a further 700,000 m^3 of the glaciolacustrine silt and clay from the surface of the spillway excavation. It will also include excavation to bedrock surface at the upstream end of the spillway approach channel where patches of silty sand and gravel exist. An estimated 225,000 m^3 of the excavated materials will be used to construct road embankment No. 1. The remainder of the excavated materials will be stockpiled for future construction of road embankment Nos. 2 and 3. Fifteen more rebound points will be installed by the Road Embankment No. 1 contractor. This portion of the spillway excavation will be completed before the Dam contractor mobilizes to the site.

d) The dyke will be constructed by the Dyke contractor from 230,000 m^3 of glaciolacustrine clay stripped from the surface of Borrow Area "A".

e) The right drainage tunnel will be driven first by the Drainage Tunnels contractor beneath the spillway to allow extensometers to be installed to record the early effects of rebound in the spillway headworks excavation. The excavation of the right drainage tunnels will be completed by Summer 1988. The installation of pressure relief drains will then take place in sequence with the grouting.

f) The Dam contractor will be allowed access only to the right abutment and river valley to the west of the river before July 1, 1988. By this date, the diversion tunnels will have been completed and the inlet and outlet stoplogs dry-tested. During the period from award until July 1, 1988, work can start on the following:

   - excavation of the right abutment for the dam and in the river valley,

   - partial excavation of the dam and cofferdam core trenches,

- partial construction of the upstream starter cofferdam,

- excavation of the remaining glaciolacustrine silt and clay, the clay till and some of the rock from the spillway headworks area and placing them in stockpiles,

- stripping the glaciolacustrine overburden from Borrow Area "A" to expose the clay till for Zone 1 excavation, and

- processing the granular materials in the river valley and terrace deposits.

g) Diversion of the river through the two 6.5 m tunnels will commence July 1988 following the Spring flood.

h) Quarrying the rock riprap and delivery and stockpiling at the site will commence during Spring 1988.

i) During the remainder of 1988, the upstream dam shell and the temporary upstream blanket will be raised to elevation 1080 m.

j) During 1988, the foundation in the dam core trench will be prepared and the grout curtain will be completed across the valley floor and on both abutments.

k) During 1988, construction of the downstream shell will commence by placing the blanket drain and some random fill from the spillway excavation.

l) During 1988, the left bank drainage tunnel will be driven commencing July 1, 1988 after the diversion tunnels have been completed. The valve equipment gallery will be excavated as an enlargement of the left drainage tunnel. Completion of the valve chamber and installation of the control valves will be done under a separate contract and will not start until mid 1990.

m) The work will be scheduled to ensure that the spillway headblock area is available by early Spring 1989 and the spillway chute and flip bucket areas are available by Summer 1989 to turn over to the Spillway contractor.

n) In 1989 the upstream shell of the dam will be raised to at least elevation 1091 m and the removal of the temporary upstream blanket and riprapping of the upstream face will commence. Fill placement in the core and downstream shell will continue.

o) The Instrumentation contract which will consist of the supply and installation of all instruments not required to be installed by the Dam or Spillway contractor will commence during 1989.

p) By the end of the 1990 construction season, the dyke, both drainage tunnels and the dam fill placement will be completed including the upstream and downstream slope protection.

q) The construction of road embankments Nos 2 and 3 and the asphalt surfacing of the Municipal District roads will not start until 1991.

The construction of the spillway will interface with the Dam contract. The Spillway contract will be awarded in the Summer of 1988. Spillway construction will be completed by December 31, 1990.

8.0 DESIGN CRITERIA AND ANALYSIS

The design of the dam, dyke, excavation slopes and other works has used analytical methods that are currently considered to be state-of-the-art and design criteria and minimum required safety factors that are consistent with those used to design similar works around the world (Wilson and Marsal (1979), USBR (1973), USCE (1970), ICOLD (1983), ICOLD (1986A), ICOLD (1986B), Sherard et al (1963), Hirschfeld and Poulos (1973), Alberta Environment (1978)).

8.1 Limit Equilibrium Analyses

The stability of the slopes of the dam and dyke and of the existing and excavated slopes were analyzed using the "PC-STABL 4" computer program for the general solution of slope stability problems by two-dimensional limit equilibrium methods. The factor of safety against instability of a slope is calculated in this program by an adaptation of the Modified Bishop and the Modified Janbu methods of slices which allow the analysis of non-circular trial failure surfaces as well as those of circular shape (Bishop (1955), Janbu (1973)). The results were checked using the Morgenstern and Price (1965) and Sarma (1973) methods.

The stability analyses were performed considering circular failure surfaces entirely within the dam as well as compound failure surfaces extending into the bedrock. Circular failure surfaces were considered for the analysis of dyke stability and excavated slopes in overburden. Compound failure surfaces only were analyzed for the excavated and existing slopes. The results of the slope stability analyses are given in detail in Appendix C and D.

TABLE XVI

MATERIAL PROPERTIES USED IN LIMIT EQUILIBRIUM ANALYSIS

	Design Effective Cohesion c' (kPa)	Design Effective Angle of Sheering Resistance (O')	Unit Weight (kN/m^3)	End-of-Construction Porewater Pressure Ratio (r_u)
Compacted Fill				
Zone 1 (impervious)	0	25°	21.6	0.4
Zone 1A: (impervious)	5	22°	21.6	0.4
Zones 2 & 3: (filter & drain	0	35°	21.6	0.0
Zones 4 & 4A: (rock fill) (1)	0	35°	21.6	0.1
Zones 5, 6, 7 & 8: (granular fill and riprap)	0	35°	21.6	0.0
In Situ Soil				
River alluvium	0	35°	21.6	0.1
River colluvium	0	25°	21.6	0.3
Glaciolacustrine silt and clay	5	22°	21.6	0.4
Glacial Till	15	25°	21.6	0.4
Bedrock-Dam Foundation				
Mudrock (cross bedding)	0	35°	23.6	0.3
Sandstone (cross bedding)	0	60°	23.6	0.2
Seam V16(2	0	17°	23.6	0.3
Seam V12(2)	0	12.5°	23.6	0.3
Bedrock - Cross Bedding				
Sandstone	0	60°	23.6	-
Intact mudrock	200	35°	23.6	-
Slightly weathered mudrock	0 to 200	35°	23.6	-
Weathered, brecciated, slickensided and badly fractured mudrock	0	35°	23.6	-

NOTES:

* (1) Zone 4A could also contain Zone 1A and 5 materials.

(2) The approximate location and other information on the seams are listed in Table III.

8.1.1 Material Properties

The material properties listed in Table III and XVI were used in the slope stability analysis of the dam, the dyke and the existing and excavated slopes and abutments.

The selection of material properties for use in the analyses has been based on laboratory testing of undisturbed soil and rock samples and samples of construction materials prepared with a compactive effort equivalent to that to be specified in the contracts. The testing methods and the rationale used to derive the design values for shear strength are described in Appendix B. The parameters selected for soil materials are consistent with those found at other sites on the Prairies (Rivard and Goodwin (1978)). The end-of-construction porewater pressure ratios were selected based on the porewater pressure response to shear stresses measured in the laboratory shear tests together with actual field experience with porewater pressure development and dissipation during the construction of existing dams of the same type and on similar foundations (USCE (1970), Hilf (1948), Bishop (1957) Li (1959), Gibson (1958), Bishop and Bjerrum (1960), Blight (1970), Gray (1972), Sherman and Clough (1968), Sherard (1963), Narita et al (1983)).

The selection of the design values of shear strength are discussed in Appendix B.

All shear strength along bedding planes in the rock between the design seams listed in Table III were represented by a design effective angle of shearing resistance of 17.5° with the exception of the zone between elevations 1085 m and 1090 m where a design effective angle of shearing resistance of 14° has been used because of the evidence that the rock is weaker within this zone.

8.1.2 Dam and Dyke Stability

All embankment slopes have been designed to have a safety factor equal to or greater than the minimum safety factors listed in Table XVII. These minimum required safety factors are consistent with those used for the design of large dams in Alberta (TransAlta (1988)) and by most authorities throughout the world (Wilson and Marsal (1979), ICOLD (1983), USCE (1970), Lauga and Taylor (1983), Binnie, Clark and Skempton (1967)). The actual safety factor computed for the dam for each of the stability cases considered are shown on the right hand side of the table as an indication of which case controls the design. The sensitivity of the factor of safety with a range of values of the design angle of shearing resistance for the controlling major seam are shown on Plates C-4 to C-10 inclusive.

Stability Cases 1 and 4 are considered normal operating conditions whereas the other cases are abnormal operating conditions.

The seams within the Basal Sandstone were found to control stability as a substantial portion of all critical slip surfaces passed through them. This is consistent with the findings of others who have designed dams on similar foundations containing seams (Ringheim (1967), Jasper and Peters (1979), BCHPA (1981), Casinader (1982), Hamel and Spencer (1984), Imrie and Meidal (1983), Imrie and Taylor (1983), Binnie, Clark and Skempton (1967), Stroman, Beene and Hull (1984), Knight (1963)).

TABLE XVII
DAM AND DYKE
SLOPE STABILITY ANALYSIS
SAFETY FACTORS

Stability Case	Required Minimum Factor of Safety	Calculated Factor of Safety (dam only)
1) Reservoir at full supply level and steady state seepage conditions	1.5	1.5 (D)*
2) Reservoir at full supply level and steady state seepage with 0.065g. horizontal pseudo-static seismic loading;	> 1.0	1.1 (D)
3) Reservoir at full supply level with steady state seepage conditions in the dam and undissipated end-of-construction porewater pressures in the foundation	1.3	1.5 (D)
4) Rapid reservoir drawdown from full supply level to minimum supply level	1.3	1.3 (U)
5) Rapid reservoir drawdown from full supply level to minimum supply level with 0.065g. horizontal pseudo-static seismic loading;	> 1.0	>1.0 (U)
6) End-of-construction porewater pressures in the dam (or dyke) and foundation	1.3	1.4 (U)
7) Reservoir at probable maximum flood level and steady state seepage conditions	1.2	1.4 (D)

* (D) means downstream slope
  (U) means upstream slope

The stability of the downstream face of the dam is governed by the long term steady state seepage Case 1, by Case 2 immediately after completion of construction and by end-of-construction Case 6. The porewater pressures in the core used for Case 1 approximate closely those derived in the seepage analysis described in Appendix E. In the foundation beneath the downstream shell of the dam, the porewater pressures will be controlled by the tailwater elevation. The critical sliding surfaces through the core of the dam for both stability cases are approximately the same; the steady state seepage Case 1 being slightly lower. Both critical sliding surfaces follow Seam V12 in Structural Unit No. 1 within the Basal Sandstone which has a design effective angle of shearing resistance of 12.5^{o}. The site investigation failed to indisputably establish that Seam V12 is in fact continuous over the entire surface of the potential sliding plane implied by the analysis. However, for the purposes of design, Seam V12 was assumed to be continuous. Similarly the stability of a sliding surface through Seam V16 in the claystone on top of the Basal Sandstone was analysed as if it was continuous even though drilling and coring in the downstream foundation of the dam failed to reveal a continuous layer of claystone overlying the Basal Sandstone in the river bottom. In many of the drill holes the river alluvium was found to lie directly on sandstone bedrock as shown on Plate B-6. This sandstone is actually the lower part of the Upper Mudrock Sequence and is not the Basal Sandstone unit as described in Section B.4.1.

A porewater pressure ratio r_u of 0.4 has been used for the plastic clay till core which will be compacted to Standard Proctor maximum density at a moisture content within one percent of optimum moisture content. Some dissipation of the construction porewater pressures is implicit in this choice of porewater pressure ratio as laboratory testing and insitu measurements during and immediately after construction of other similar dams have shown this to be reasonable (Gibson (1958), Sherard et al (1963), USCE (1970), Bishop and Bjerrum (1960), Sherman and Clough (1968)). The porewater pressure ratio r_u of 0.3 in the dam foundation allows for the short drainage path that will exist from the thin claystone layer containing the seam, to the overlying pervious river alluvium and to the underlying jointed sandstone.

The effect of different rates of dissipation of porewater pressures in the foundation seam compared with those in the dam core was analyzed in Case 3. This is of interest during the period when the porewater pressure regime changes from the end-of-construction conditions to those obtaining during steady state seepage conditions with the reservoir at full supply level. Porewater pressure ratios r_u of 0.3, 0.2 and 0.15 were used to simulate different levels of porewater pressure in the foundation seam that may exist due to dissipation. If no change takes place within the seam and r_u does not reduce due to disipation effects from the value of 0.3 at the end of construction the safety factor would be 1.5 which is higher than the required minimum of 1.3. Some porewater dissipation must occur due to drainage during the one or more years it will take for the reservoir to fill and so the safety factor will be greater than 1.5.

The reservoir will be drawn down annually to different levels depending on the downstream demand and on the inflow. The maximum depth of drawdown will be from full supply level to minimum supply level at elevation 1064 m.

In Case 4 it has been assumed that the 54.6 m of drawdown will occur instantaneously with respect to dissipation of porewater pressures in the rockfill of the upstream shell of the dam. The upstream layer of Zone 5 granular fill was assumed to be free draining compared to the rate of reservoir drawdown. For this stability case the safety factor of the upstream slope will be the required minimum of 1.3. This is conservative as the actual drawdown will take place over a 3 to 6 month period during which some dissipation of porewater pressures must occur (Morgenstern (1963)). Case 4 together with Case 6 govern the stability of the upstream slope of the dam. Case 5 could also be considered to govern but the probability of an earthquake occurring at the time that undissipated porewater pressures are present following full reservoir drawdown is very low.

Case 2 considers the effect of an earthquake on the downstream slope in terms of limiting equilibrium (Trans Alta (1988)). The results must be considered in conjunction with the results of the dynamic analyses discussed in Section 8.2 as such limit equilibrium analyses only approximate the performance of the dam under seismic loading.

Case 7 demonstrates that even if there was sufficient time for steady state seepage porewater pressures to develop in the core, during the few days that it will take for the reservoir to surcharge to the PMF level, the safety factor would only drop to 1.4. This is acceptable for such an extreme condition.

The critical sliding surface for upstream and downstream slope stability passes through the core and Seam V12. Plate C-2 in Appendix C lists the calculated safety factors for a sliding surface passing through both Seam V12 and Seam V16, which have been assumed to be continuous along the critical sliding surface for the purposes of the analysis.

Detailed results of the stability analyses are presented in Appendix C.

8.1.3 Stability of the Excavated and Existing Slopes

The stability of the river banks in the vicinity of the dam and the slopes for the spillway excavation were analyzed using the design seams listed in Table III, the cross-bedding material properties listed at the bottom of Table XVI and the required safety factors listed below in Table XVIII.

The stabilizing effect of the three dimensional shape of an actual sliding mass and the actual location of horizontal seams relative to the shape of the excavated slopes was analyzed (ICOLD (1986a)). These indicate that the factors of safety calculated from the two dimensional analysis are generally conservative.

The detailed results of the stability analyses on the excavated and existing slopes are presented in Appendix D, together with sensitivity analyses showing the effect of varying shear strength of seams on safety factors.

a) Spillway Excavation

The excavated slopes shown on Plate 9 have been based on a two dimensional analyses considering the presence of the major seams listed in Table III.

In all cases analyzed the critical sliding surface passes down to and along a sub-horizontal seam near the base with a dip of 0.6° to the north. Each slope was analyzed separately. A simplified stratigraphy was developed for each slope based on adjacent drill hole and surface exposure logs. The cross-bedding shear strength parameters for each rock type are listed in Table XVI. The amount of effective cohesion used depended on the degree of weathering and the claystone content of the mudrock recorded in the logs. The maximum 200 kPa was used for intact mudrock with a high claystone content. No effective cohesion was allocated to highly weathered, brecciated, slickensided or fractured mudrock. Values between 0 and 200 kPa were used for intermediate conditions.

TABLE XVIII

EXCAVATED AND EXISTING SLOPES SAFETY FACTORS

	Stability Case	Required Minimum Factor of Safety
1.	Reservoir at full supply level and steady state seepage conditions.	1.5
2.	Reservoir at full supply level and steady state seepage with horizontal pseudo-static seismic loading 0.065 g.	> 1.0
3.	Rapid reservoir drawdown from full supply level to minimum supply level.	1.3
4.	Rapid reservoir drawdown from full supply level to minimum supply level with horizontal pseudo-static seismic loading 0.065 g (for upstream slopes only).	> 1.0

Sandstone layers which are known to be jointed will act as drains and a corresponding phreatic surface was introduced into the analysis.

The analysis of the rock conditions in the Upper Mudrock, that are evidenced in the spillway shaft, the right bank drillholes and laboratory shear box test data, is described in Appendix B.

The spillway approach channel slopes will be excavated to one horizontal to one vertical in rock with a 10 m wide berm at elevation 1120 m and a 10 m wide berm at the rock-overburden contact and a slope of three horizontal to one vertical in the overburden above. All the stability cases have factors of safety higher than, or equal to the required minimum factors of safety.

Three sections were analyzed along the spillway chute excavated slopes. In these sections, 0.6° dip of the rock bedding to the north was used. The analyses dictate that the chute side slopes should be excavated to one horizontal to one vertical in rock, with a 10 m wide berm every 20 m height from the base of the excavation, with a 10 m wide berm at the rock-overburden contact, and a slope of three horizontal to one vertical in the overburden above. Except for the slopes in the region of 50 m downstream of the spillway headworks structure, the calculated safety factors for all stability cases, based on a two dimensional analysis, are higher than, or equal to the required minimum factors of safety.

For the slopes in the region of 50 m downstream of the spillway headworks the calculated safety factor for the normal long term condition based on a two dimensional analysis is about seven percent less than the required minimum. The design is considered acceptable because analyses comparing two and three dimensional methods indicate that for the following reasons the actual safety factor will be higher than the required minimum. This is discussed in Appendix C.

i) The spillway chute is inclined at a slope of 6.4 horizontal to one vertical along its axis and at the base of the excavated slope. The seams are essentially horizontal and any potential shearing surface will pass through them and will mobilize higher cross bedding shear strength at the upstream end where the seam lies below the bottom of the excavation. As ground surface also slopes downwards in the same direction as the spillway chute excavation, the seam effectively daylights downstream. These constraints will limit the lateral extent of any potential shearing surface and will cause the actual safety factor to be greater than that calculated.

ii) The cohesion present in the mudrock will also cause a potential shearing surface to be restrained laterally.

iii) The phreatic surface assumed in the analyses is high if the combined effect of the grout curtain, the line of pressure relief drains, the drainage blanket beaneath the chute slab and river slope on the back side of the excavated slope are taken into account.

b) Dam Abutment

The results of the two dimensional slope stability analyses indicates that the soil slopes upstream of the dam axis on the left bank should be excavated to a slope of four horizontal to one vertical to achieve the required minimum factor of safety of 1.3 for the rapid reservoir drawdown case. For these slopes, all other stability cases have factors of safety higher than, or equal to the required minimum factors of safety.

The existing natural rock slope on the left bank upstream of the dam axis, during rapid reservoir drawdown from full supply level top minimum supply level, has a computed safety factor equal to 1.2 which is eight percent less than 1.3, the required minimum factor of safety. This slope will be stable because of the three dimensional effects. For all other stability cases the factors of safety for this slope are higher than the equal to, or required minimum factors of safety.

The overburden slope on the right bank upstream of the dam axis will be excavated to 3.5 horizontal to one vertical to achieve the required minimum factor of safety of 1.5 for the steady state seepage condition with the reservoir at full supply level. For this slope, all other stability cases have factors of safety higher than, or equal to the required minimum factor of safety.

The soil slopes on both right and left bank downstream of the dam axis will be excavated to three horizontal to one vertical to achieve the required minimum factor of safety of 1.5 for the steady stage seepage condition with the reservoir at full supply level. For these slopes, all other stability cases have factors of safety higher than, or equal to the required minimum factors of safety.

For the existing natural rock slope on the right bank downstream of the dam axis, all stability cases have factors of safety higher than, or equal to the required minimum factors of safety.

8.2 Finite Element Analysis

Static and dynamic finite element analyses were performed on the dam to determine the stability of the structure under static and earthquake dynamic loadings (ICOLD (1983), ICOLD (1986a), ICOLD (1986), Wilson and Marsal (1979)).

The finite element static stress analysis of the dam and its foundation was used to determine the stress and strain distribution. This analysis provided an initial estimate of the static stress distribution and the anticipated material deformation characteristics. The results were used to assess the stability of the dam and to develop the initial stress distribution to be input into the dynamic stress analysis.

The analysis of dynamic stresses and displacements developed in the dam when subjected to the design earthquake accelerations was used to compare the stresses with available material strengths and to compare the displacements with allowable maximum deformations.

8.2.1 Analytical Models

Two separate nonlinear finite element models were used in the analysis. The pre-earthquake stress condition was evaluated using the nonlinear finite element program "SAN" developed by Davachi (1978) at the Imperial College, University of London, UK. This program is a nonlinear two-dimensional finite element model which uses "gravity turn-on" loading by simulating the construction of the dam in layers utilizing the dense liquid technique. The dynamic stress analysis was conducted using the two-dimensional finite element program "QUAD4" developed by Idriss et al (1973) at the Earthquake Engineering Research Centre, University of California, Berkeley. The acceleration-time relationship used was the 1952 Taft earthquake spectrum normalized to a peak acceleration of 0.13g (Wolofsky and Gough (1985)).

8.2.2 Material Properties

Table XIX lists the material properties that were used in the static analysis and Table XX lists the initial values used in dynamic analysis.

With the exception of the bedrock units, all materials were assigned initial shear modulus values based upon site specific laboratory cyclic test data. The values are consistent with those recommended by Seed and Idriss (1970).

TABLE XIX

MATERIAL PROPERTIES USED IN FINITE ELEMENT STATIC ANALYSIS (1)

	Effective Cohesion c' (kPa)	Effective Angle of Shearing Resistance o'	Unit Weight (kN/m^3)	Poissons Ratio	E_i (MPa)
Foundation Materials					
1. Colluvium and Alluvium	0	28°	21.6	0.3	84 + 267 (m)(2)
2. Intact Mudrock	--	Linear	23.6	0.4	2,000
3. Intact Sandstone	--	Linear	23.6	0.3	17,000
4. Seam V16	0	17°	23.6	0.4	400
5. Seam V12	0	12.5°	23.6	0.3	400
Compacted Fill Materials					
6. Core (Zone 1)	0	25°	19.6	0.4	20 + 67 (m)
7. Shells (Zones 2 to 8)	0	35°	21.6	0.3	84 + 267 (m)

Notes:

(1) The material properties used in the finite element analyses were selected during the early stages of the site investigation and differ slightly from those listed in Table XVI which are based on all of the site data. It was not considered necessary to re-compute the finite element analyses after the material properties were modified as the differences would have little effect on the results.

(2) m = effective mean principal stress

The material properties for the bedrock units are based on static triaxial and direct shear box laboratory tests on samples taken from the site.

The material properties used in the dynamic analyses are based on values taken from the literature and the results of cyclic laboratory testing of samples from the site. The laboratory testing also showed that there are no losses in the strength of compacted till or in the residual angle of shearing resistance of the seams under dynamic loading. The laboratory testing results are discussed in Appendix B.

The details and results of the finite element analysis and the dynamic analysis are presented in Appendix C.

Conclusions from the static analysis are given in Sections 8.9 and 8.10. The dynamic response analysis showed that the maximum displacement during an earthquake would be 57 mm and the maximum final plastic displacemnt would be 21 mm. These are acceptable deformations because the dam could sustain them and continue to function without any reduction in the margin of safety against failure.

8.3 Settlement

The settlement that will occur in the dam and dyke fill, in the overburden, and in the foundation bedrock was assessed analytically using consolidation data obtained in the laboratory. The experience of others was also considered. This experience is documented in the literature of the actual as compared with calculated settlement of existing dams (Sherard (1963), Skermer (1975), Wilson (1973), Eisenstein and Law (1977), Kulhawy and Duncan (1972)).

TABLE XX
INITIAL MATERIAL PROPERTIES INPUT TO DYNAMIC ANALYSIS(1)

	Unit Weight (lb/ft^3)(2)	K_2	Shear Modulus (k/ft^2)(G_{max})	Poissons Ratio
Foundation Materials				
1. Colluvium and Alluvium	137	40	--	0.3
2. Intact Mudrock	150	--	13,900	0.4
3. Intact Sandstone	150	--	118,000	0.3
4. Seam V16	150	50	--	0.4
5. Seam V12	150	50	--	0.3
Compacted Fill Materials				
6. Core (Zone 1)	125	50	--	0.4
7. Shells (Zones 2 to 8)	137	75	--	0.3

Note:

(1) The material properties used in the finite element analyses were selected during the early stages of the site investigation and differ slightly from those listed in Table XVI which are based on all of the site data. It was not considered necessary to re-compute the finite element analyses after the material properties were modified as the differences would have little effect on the results.

(2) "QUAD4" is written in the Imperial system of units.

A significant portion of the settlement within the fill and the overburden, and some of the settlement in the bedrock will occur during construction. Most of the bedrock settlement will be as a result of the claystone layers consolidating. The balance of the consolidation settlement will occur over a longer period of time and will require the crest of the dam to be built above the nominal required elevation. This will be accommodated by locally steepening the top of the dam and dyke slopes. The settlement allowance specified allows for the smaller amount of settlement that will occur in the abutments and takes into account previous experience that has shown that the magnitude of calculated consolidation settlements for over-consolidated soils and soft rocks are rarely realized in practice. A maximum settlement of 1 m has been estimated for the design.

8.4 Seepage Analysis

Seepage through and under the dam and dyke, as well as seepage and exit gradients, were analyzed using a finite element analysis of seepage flow through porous media (Sherard (1963)).

Detailed consideration was given to the seepage and hydraulic gradients downstream of the core-bedrock interface and through the grout curtain. The results of the analysis were used to determine the steady state seepage pore pressures to be used in the stability analyses of the dam to determine the most effective location of the drainage tunnel and grout curtain, and to investigate the hydraulic gradients that might exist close to the drainage tunnels. The results of the seepage analysis are presented in Appendix E.

8.5 Freeboard Analysis

The crest elevation of 1125.5 m of the dam and dyke was established by providing freeboard of 1 m above the surcharged reservoir level during the probable maximum flood. This exceeds the requirement of Alberta Dam SAfety Guidelines (Alberta Environment (1978)). The freeboard above full supply level is 6.9 m. The adequacy of this freeboard under the action of wind on the reservoir producing wind setup, waves and wave run-up was checked using the 1:1000 year wind with the reservoir at full supply level. The reservoir will be operated at full supply level during all floods less than the 1:1000 year project design flood. It was found that maximum required freeboard against waves and wind action is 2.2 m above FSL.

Maximum hourly over-water wind velocities for the above analyses were computed from recorded data obtained from the Atmospheric Environmental Service meteorological station at Pincher Creek, Alberta approximately 10 km southwest of the project site.

8.6 Upstream Slope Protection

The upstream slope of the dam and dyke will be protected against the action of waves and wave run-up generated by the wind action (USCE (1982), Sherard et al (1963)).

The design of the riprap and riprap bedding was based on the US Army Coastal Engineering Research Center, Shore Protection Manual (USCE (1984)). A summary of the results of the riprap design is included in Appendix F.

As downstream demands on the reservoir increase in the future, the reservoir will be operated throughout the entire range of reservoir levels. The slope protection was therefore extended over the entire upstream face of the dam and dyke.

A number of possible reservoir operation scenarios was used to divide the upstream slope of the dam into zones requiring different wave erosion protection. The probability of the reservoir level being at any given elevation at or below full supply level was combined with the probability of the wind speed to obtain a combined probability of a 1:100 year event.

The riprap size required to protect each part of the upstream slope of the dam against wave action was established using the wind speed appropriate to the combined probability of 1:100 years and the effective fetch at that reservoir elevation.

The stabilization of the river banks adjacent to the dam that will be exposed to wave attack will include riprap slope protection of the soil portion of the slopes. The slope treatment of the abutments of the dam, land upstream of the dyke and the specified development of Borrow Area "A" has been based on the assumption that the glaciolacustrine silt and clay and, to a lesser extent the underlying clay till, will ultimately erode, under the action of reservoir waves, to a stable slope of 15 horizontal to one vertical.

8.7 Evaluation of Liquefaction Potential of Foundation Alluvium and Colluvium

The liquefaction potential of the river alluvium and colluvium was evaluated from the results of in situ density tests and cyclic laboratory tests and the correlation between earthquake shaking conditions causing liquefaction or cyclic mobility and the standard penetration test in these materials (Seed (1979)). The results are presented in Appendix C. It was concluded that all silty river alluvium and colluvium with less than 50 percent gravel size particles and an in situ dry density less than 95 percent of Standard Proctor maximum dry density must be removed from the dam foundation.

8.8 Filter Design

Filter design was based on the following criteria (Wilson and Marsal (1979), USCE (1982), Sherard (1963)).

To avoid movement of particles from the base:

a) The 15 percent size of the filter material will be equal to or less than 5 times the 85 percent size of the material being protected (base material).

b) The 50 percent size of the filter material will be equal to or less than 25 times the 50 percent size of the base material.

c) The 15 percent size of the filter material will be equal to or less than 20 times the 15 percent size of the base material.

d) For widely graded base materials, the 15 percent size of the filter material will be equal to or less than 40 times the 15 percent size of the base material.

e) For the upstream slope protection, the 15 percent size of the riprap will be equal to or less than 5 times the 85 percent size of the riprap bedding.

To avoid excess head loss in the filter the 15 percent size of the filter material will be equal to or greater than 5 times the 15 percent size of the base material.

To avoid movement of filter into drain pipe perforations or joints:

a) The 85 percent size of the filter material will be equal to or greater than 1.2 times the slot width.

b) The 85 percent size of the filter material will be greater than the diameter of the hole.

Filters where the base material contains a large percentage of gravels, were designed on the basis of the gradation curve of the base material which is finer than the 1 mm size.

Internal stability of the filter and drain materials was also checked (Kenney et al (1985)).

The thickness of the downstream drainage blanket was determined by assuming a minimum coefficient of permeability of 1 cm/s for the Zone 3 Drain material, and confined flow with a hydraulic gradient of not more than 0.02. A safety factor of at least 100 was then applied against the estimated seepage flow through, and beneath the dam based on the seepage analyses described in Appendix E. This requirement indicated that a 2 m thick layer of Zone 3 Drain material would be conservative and consistent with current dam design practice.

8.9 Hydraulic Fracturing Analysis

The possibility of hydraulic fracturing of the core of the dam was analyzed using the static finite element program "SAN" (Sherard et al (1963)). The material properties given in Table XIX were used in the analysis. The definition of the factor of safety against hydraulic fracturing was based on the procedures given by Nobari et al (1971), Vaughan (1976) and Davachi (1978). The results of the analysis are presented in Appendix C.

Although it was calculated that hydraulic fracturing will not occur even when the reservoir level is surcharged during a flood larger than the 1000 year event, safeguards against hydraulic fracturing and piping have been included in the design of the internal zones of the dam. These safeguards are:

- the use of a gradation for Zone 2 Filter material that is effectively concrete sand aggregate, as this has been found by experience to prevent particle migration from cracks in fine grained core materials,
- strict adherence to the filter criteria in the design of Zone 2 Filter and Zone 3 Drain gradations,

- the use of a wide filter zone,
- the use of a wide dam core, and
- using a higher placement moisture content for the dam core adjacent to the foundation and abutment contact, and in the upper part of the dam.

8.10 Core-Shell Interaction

The load transfer between shell and core and core-shell interaction was analyzed using the static finite element program "SAN" and was found to be within acceptable limits. The material properties given in Table XIX were used in the analysis. A parametric study was carried out to analyze the amount of load transfer from core to shells for different stiffness ratios of core and shell material. The results are presented in Appendix C.

8.11 Road Embankments and Permanent Road

The five road embankments will be homogeneous earth fills constructed of the glaciolacustine clay that will be removed from the spillway excavation and from the surface of Borrow Area "A" to expose the clay till to be used in the dam core. As most of this glaciolacustine clay would otherwise be wasted, the road embankments have been designed with conservatively flat slopes that conform to Alberta Transportation requirements for road fills without guard rails. The parts of the permanent road across the top of the dam, dyke and spillway were designed to Alberta Transportation standards.

9.0 INSTRUMENTATION

The dam and dyke will be monitored during and after construction with instrumentation to ensure that:

- the parameters used in the design are applicable,
- the short-term performance during construction and after initial reservoir filling is acceptable,
- long-term effects of seepage, settlement deformation and wave action are not detrimental to the integrity of the dam, and
- the need for maintenance or remedial treatment is identified early.

To meet these requirements, the following instrumentation will be installed in the dam and its foundation:

a) Pneumatic, vibrating wire, double tube hydraulic and Casagrande type piezometers located in the core, in the foundation claystone and in the abutment slopes to measure:

   - build-up of pore water pressure during construction and rate of dissipation after construction,
   - pore water pressures during reservoir filling, and
   - seepage gradients,

b) Surface settlement monuments located at regular intervals along the crest and benches to measure horizontal and vertical movements,

c) Slope indicators to measure the internal deformation of the core and shells,

d) Settlement profilers to monitor the distribution of settlement with depth both in the dam and within the foundation,

e) Horizontal movement indicators to monitor any horizontal movements during and after construction and during reservoir filling,

f) Extensometers in the right drainage tunnel to measure heave of the bottom of the spillway excavation,

g) Rebound gauges embedded in the rock before the start of spillway excavation to measure by deep levelling methods the rebound of the spillway excavation due to unloading, and

h) Seepage beneath the dam and through the dam abutments that passes into the downstream drainage blanket and surfaces at the toe of the dam will be channelled by low dykes and passed over measuring weirs located downstream of the toe of the dam. The seepage into the downstream drainage blanket from both abutments will be separated from the seepage in the river valley by constructing two vertical impervious barriers through the downstream drainage blanket perpendicular to the dam axis.

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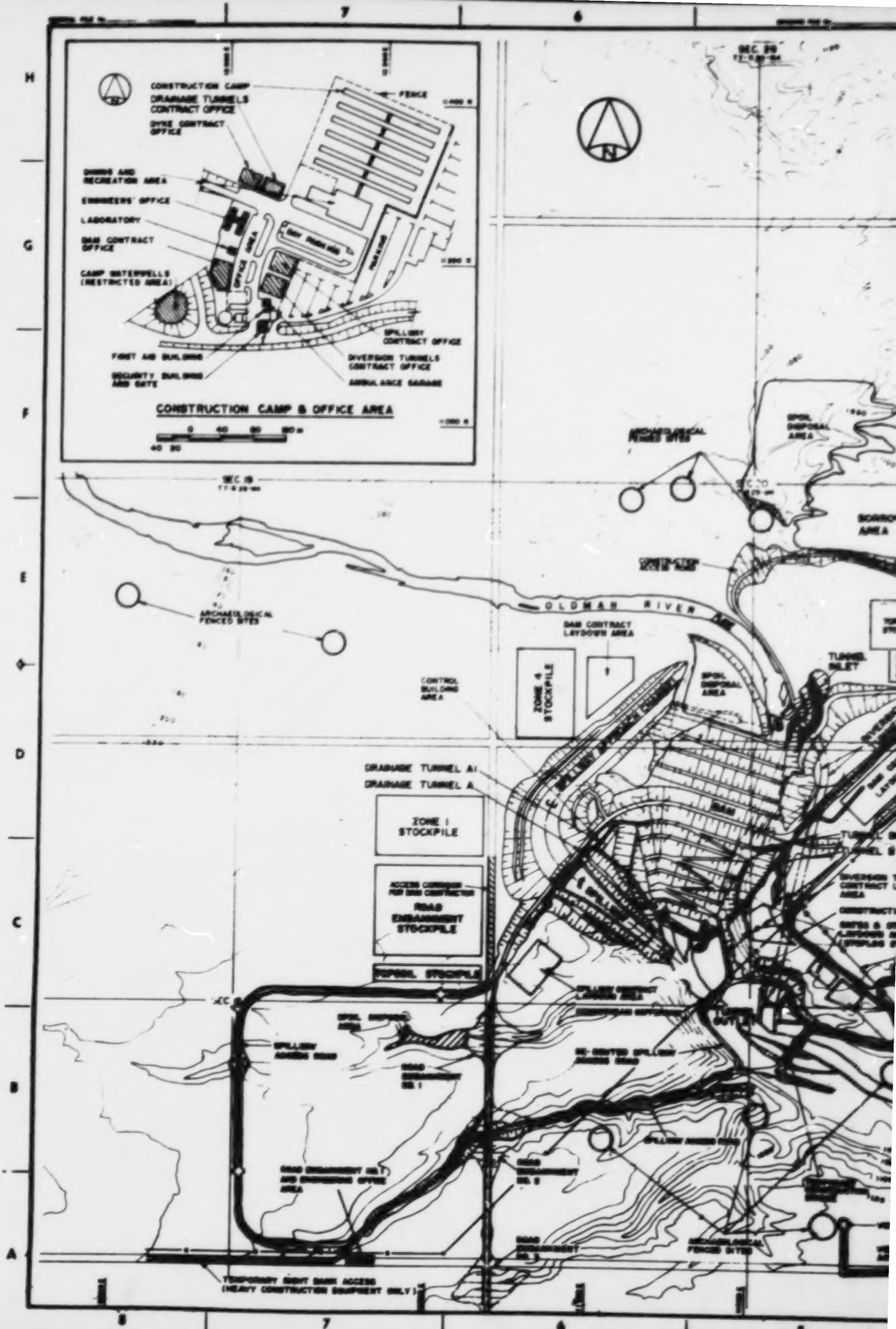
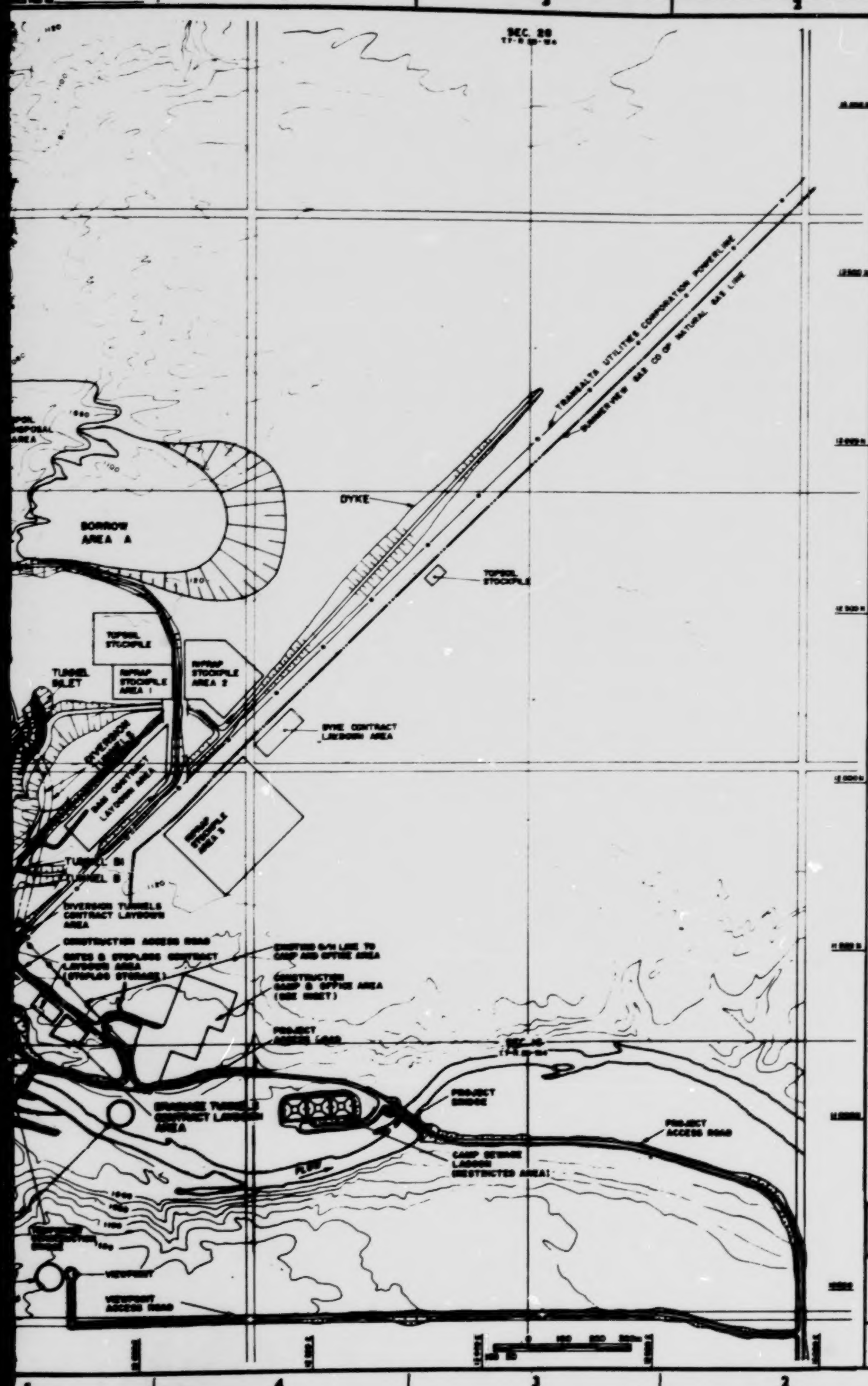
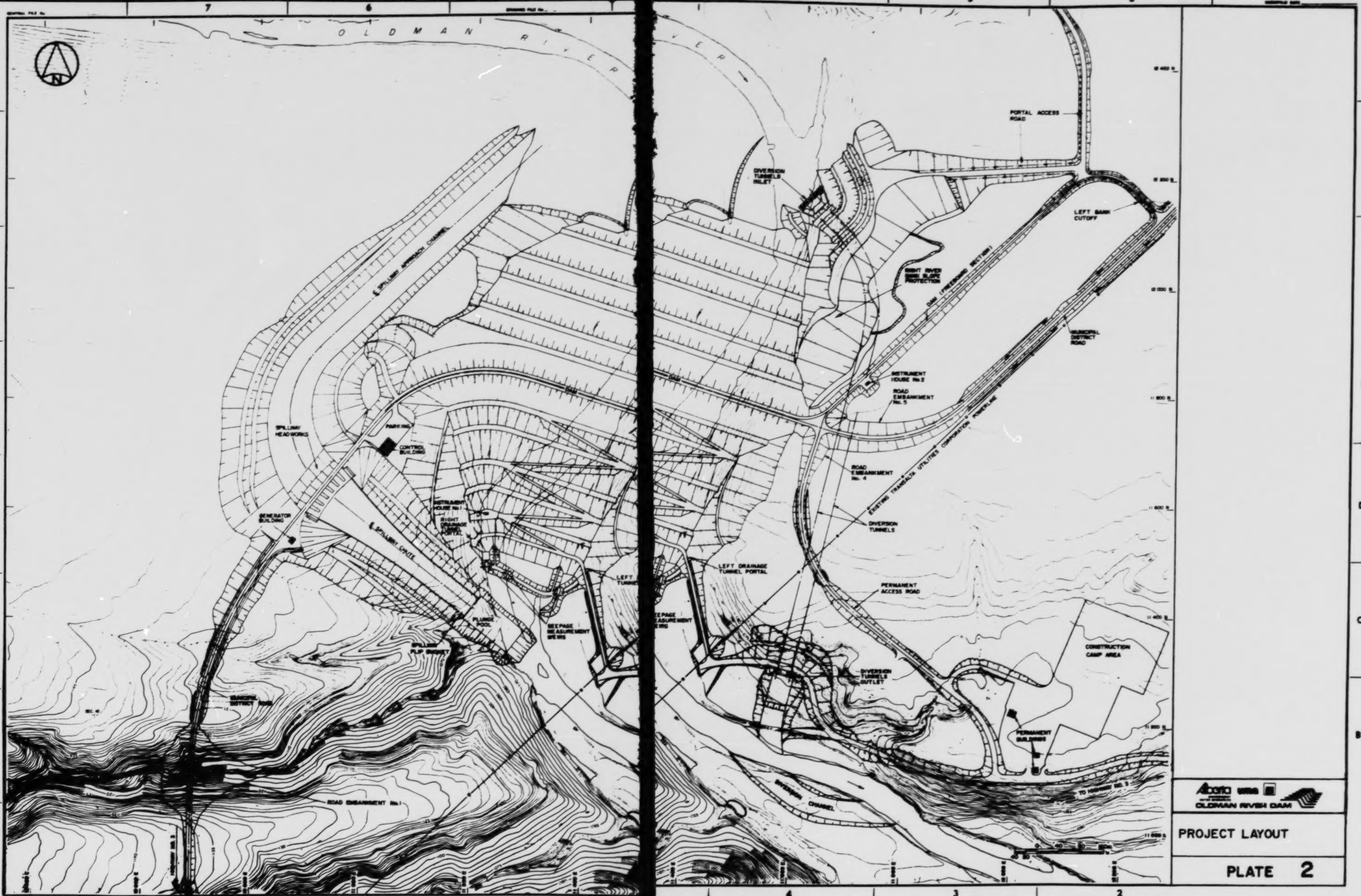

CONSTRUCTION CAMP
DRAINAGE TUNNELS CONTRACT OFFICE
DYKE CONTRACT OFFICE
FENCE
DINING AND RECREATION AREA
ENGINEERS' OFFICE
LABORATORY
DAM CONTRACT OFFICE
CAMP WATERWELLS (RESTRICTED AREA)
OFFICE AREA
PARKING
FIRST AID BUILDING
SECURITY BUILDING AND GATE
SPILLWAY CONTRACT OFFICE
DIVERSION TUNNELS CONTRACT OFFICE
AMBULANCE GARAGE
CONSTRUCTION CAMP & OFFICE AREA
0 40 80 120 m
40 20
SEC 19
SEC 20
SEC 29
ARCHAEOLOGICAL FENCED SITES
SPOIL DISPOSAL AREA
BORROW AREA
CONSTRUCTION ACCESS ROAD
OLDMAN RIVER
DAM CONTRACT LAYDOWN AREA
ZONE 4 STOCKPILE
CONTROL BUILDING AREA
TUNNEL INLET
DRAINAGE TUNNEL A1
DRAINAGE TUNNEL A
ZONE I STOCKPILE
ACCESS CORRIDOR FOR DAM CONTRACTOR
ROAD EMBANKMENT STOCKPILE
TOPSOIL STOCKPILE
DAM
SPILLWAY ACCESS ROAD
ROAD EMBANKMENT NO. 1
ROAD EMBANKMENT NO. 2
ROAD EMBANKMENT NO. 3
ROAD EMBANKMENT NO. 1 AND ENGINEERING OFFICE AREA
RE-ROUTED SPILLWAY ACCESS ROAD
DIVERSION TUNNELS CONTRACT LAYDOWN AREA
TEMPORARY RIGHT BANK ACCESS (HEAVY CONSTRUCTION EQUIPMENT ONLY)
H
G
F
E
D
C
B
A
8
7
6
5


SEC. 29
DYKE
TRANSALTA UTILITIES CORPORATION POWERLINE
PINCHER CREEK GAS CO-OP NATURAL GAS LINE
BORROW AREA A
TOPSOIL DISPOSAL AREA
TOPSOIL STOCKPILE
RIPRAP STOCKPILE AREA 1
RIPRAP STOCKPILE AREA 2
RIPRAP STOCKPILE AREA 3
TUNNEL INLET
DIVERSION TUNNELS
DAM CONTRACT LAYDOWN AREA
DYKE CONTRACT LAYDOWN AREA
TUNNEL B1
TUNNEL B
DIVERSION TUNNELS CONTRACT LAYDOWN AREA
CONSTRUCTION ACCESS ROAD
GATES & STOPLOGS CONTRACT LAYDOWN AREA (STOPLOG STORAGE)
EXISTING R/W LINE TO CAMP AND OFFICE AREA
CONSTRUCTION CAMP & OFFICE AREA (SEE SHEET)
PROJECT ACCESS ROAD
DRAINAGE TUNNELS CONTRACT LAYDOWN AREA
PROJECT BRIDGE
CAMP SEWAGE LAGOON (RESTRICTED AREA)
FLOW
PROJECT ACCESS ROAD
VIEWPOINT
VIEWPOINT ACCESS ROAD
1100
1120
1080
4
3
2
5

OLDMAN RIVER DAM

PROJECT SITE

PLATE 1


OLDMAN RIVER
N
℄ SPILLWAY APPROACH CHANNEL
SPILLWAY HEADWORKS
PARKING
CONTROL BUILDING
GENERATOR BUILDING
℄ SPILLWAY CHUTE
INSTRUMENT HOUSE No.1
RIGHT DRAINAGE TUNNEL PORTAL
PLUNGE POOL
SPILLWAY FLIP BUCKET
SEEPAGE MEASUREMENT WEIRS
MUNICIPAL DISTRICT ROAD
ROAD EMBANKMENT No.1
HIGHWAY No. 3
DAM
DIVERSION TUNNELS INLET
PORTAL ACCESS ROAD
LEFT BANK CUTOFF
RIGHT RIVER BANK SLOPE PROTECTION
DAM (FREEBOARD SECTION)
MUNICIPAL DISTRICT ROAD
INSTRUMENT HOUSE No.2
ROAD EMBANKMENT No.5
ROAD EMBANKMENT No.4
EXISTING TRANSALTA UTILITIES CORPORATION POWERLINE
DIVERSION TUNNELS
LEFT DRAINAGE TUNNEL PORTAL
LEFT TUNNEL
PERMANENT ACCESS ROAD
SEEPAGE MEASUREMENT WEIRS
DIVERSION TUNNELS OUTLET
DIVERSION CHANNEL
CONSTRUCTION CAMP AREA
PERMANENT BUILDINGS
TO HIGHWAY No. 3
12 400 N
12 200 N
12 000 N
11 800 N
11 600 N
11 400 N
11 200 N
11 000 N
H
G
F
E
D
C
B
A
8
7
6
5
4
3
2
1

OLDMAN RIVER DAM

PROJECT LAYOUT

PLATE 2

ELEVATION (m)
1130
1110
1090
1070
1050
1030
FULL SUPPLY
MSL EL. 1064
FULL SUPPLY LEVEL EL. 1118.6
PMF LEVEL 1124.5
MSL EL. 1064
DAM AXIS
NOMINAL CREST EL. 1125.5
EL. 1110
EL. 1100
EL. 1085
EL. 1080
EL. 1075
EL. 1065
EL. 1060
EL. 1080
EL. 1105
EL. 1058
IMPERVIOUS BLANKET TO BE REMOVED AND REPLACED WITH RIPRAP AFTER COMPLETION OF CORE TO EL. 1080
EXISTING ROCK FILL PLACED AS PART OF DIVERSION TUNNELS CONTRACT
25
60
0.67
0.25
3(TYP)
1.5
3.5
10 MIN
9
4
6
11
①
②
③
④
④A
④B
⑤
⑦
⑧A
⑧B
⑧C
⑧D

LEGEN
① CO
ⒶIM
② FI
③ DR
④ RO
④A UN
④B OV

LEGEND:

① CORE
①A IMPERVIOUS FILL
② FILTER
③ DRAIN
④ ROCKFILL
④A UNCLASSIFIED FILL
④B OVERSIZE ROCKFILL

⑤ GRANULAR FILL
⑥ PERMANENT ROAD GRAVEL
⑦ RIPRAP BEDDING
⑧A RIPRAP CLASS 'A'
⑧B RIPRAP CLASS 'B'
⑧C RIPRAP CLASS 'C'
⑧D RIPRAP CLASS 'D'

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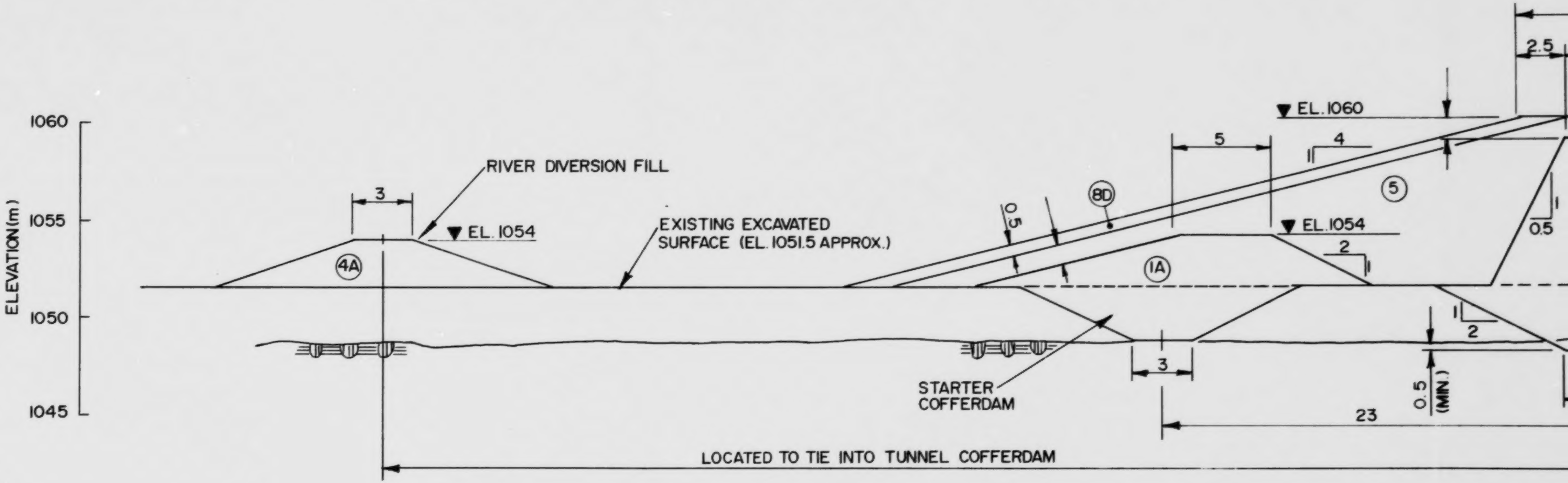
OLDMAN RIVER DAM

DAM SECTION

PLATE 3


1060
1055
1050
1045
ELEVATION(m)
RIVER DIVERSION FILL
3
EL. 1054
4A
EXISTING EXCAVATED
SURFACE (EL. 1051.5 APPROX.)
0.5
8D
5
4
1
EL. 1060
2.5
5
EL. 1054
2
1
1A
1
0.5
1
2
3
STARTER
COFFERDAM
0.5
(MIN.)
23
LOCATED TO TIE INTO TUNNEL COFFERDAM

LEGEND:

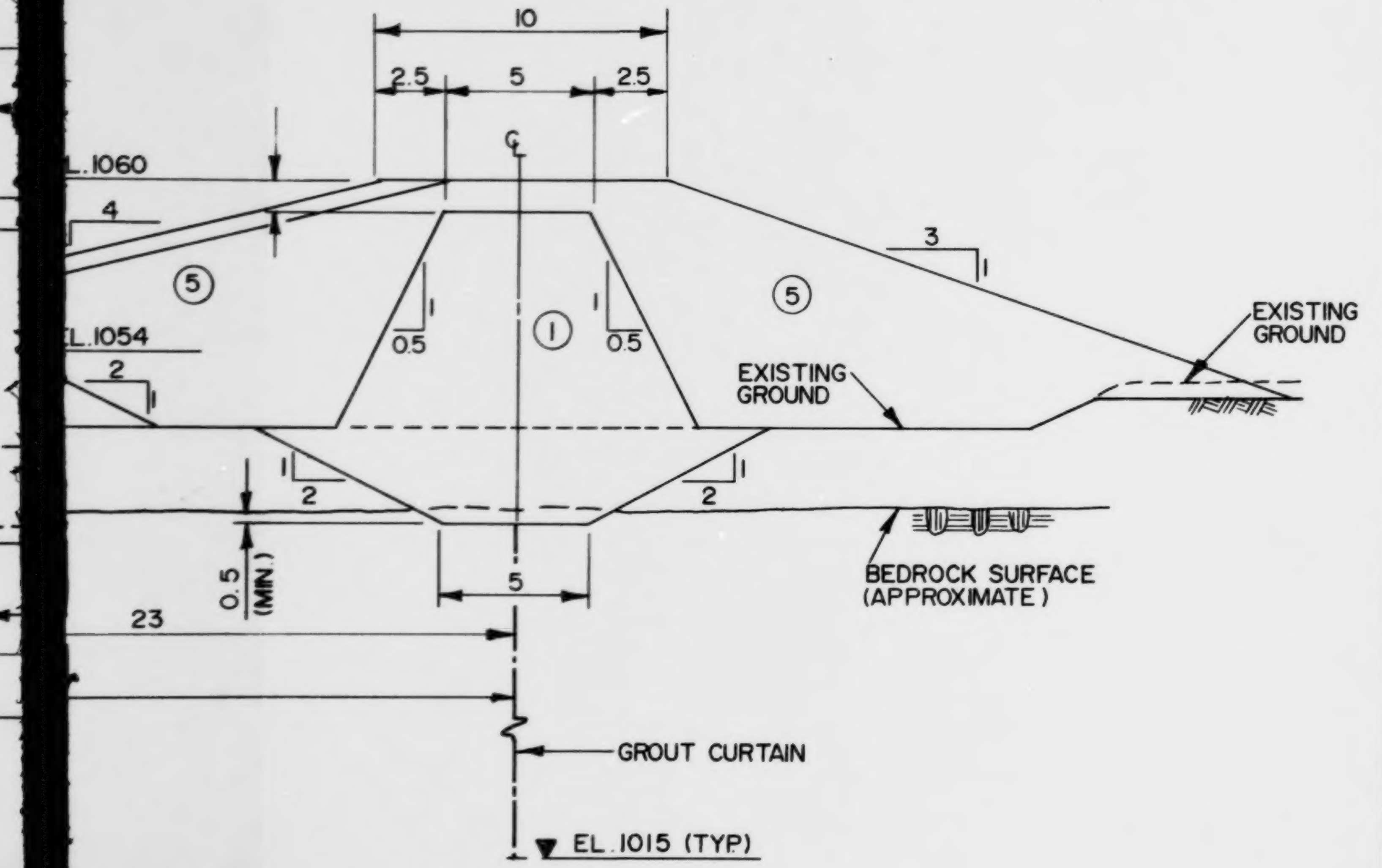
(1) CORE

(1A) UNCLASSIFIED IMPERVIOUS FILL

(4A) UNCLASSIFIED FILL

(5) GRANULAR FILL

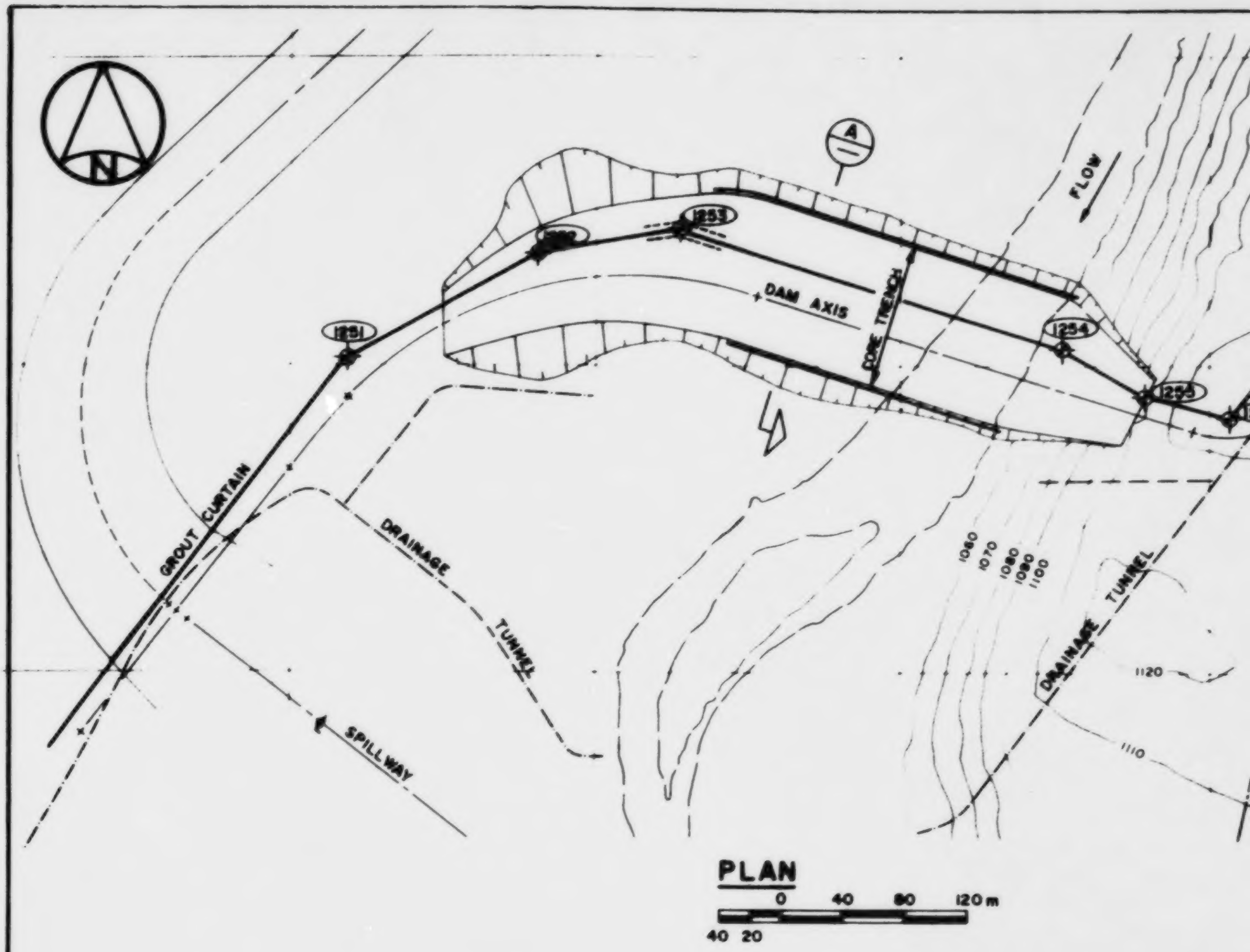
(8D) RIPRAP CLASS 'D'


10
2.5
5
2.5
℄
L.1060
4
⑤
⑤
3
1
1
0.5
①
1
0.5
EXISTING GROUND
L.1054
2
1
EXISTING GROUND
1
2
2
1
0.5 (MIN.)
5
BEDROCK SURFACE (APPROXIMATE)
23
GROUT CURTAIN
EL.1015 (TYP.)

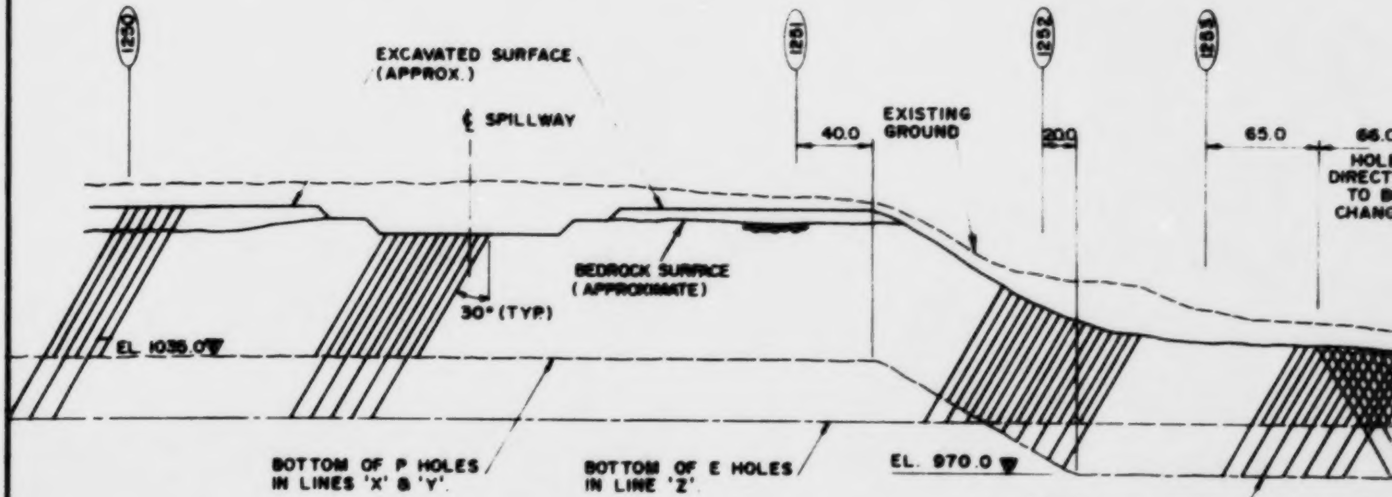
Alberta uma
OLDMAN RIVER DAM

COFFERDAM SECTION

PLATE 4


A
FLOW
1252
1253
1251
DAM AXIS
CORE TRENCH
1254
1255
GROUT CURTAIN
DRAINAGE TUNNEL
℄ SPILLWAY
1060
1070
1080
1090
1100
DRAINAGE TUNNEL
1120
1110
0
40
80
120 m
40
20

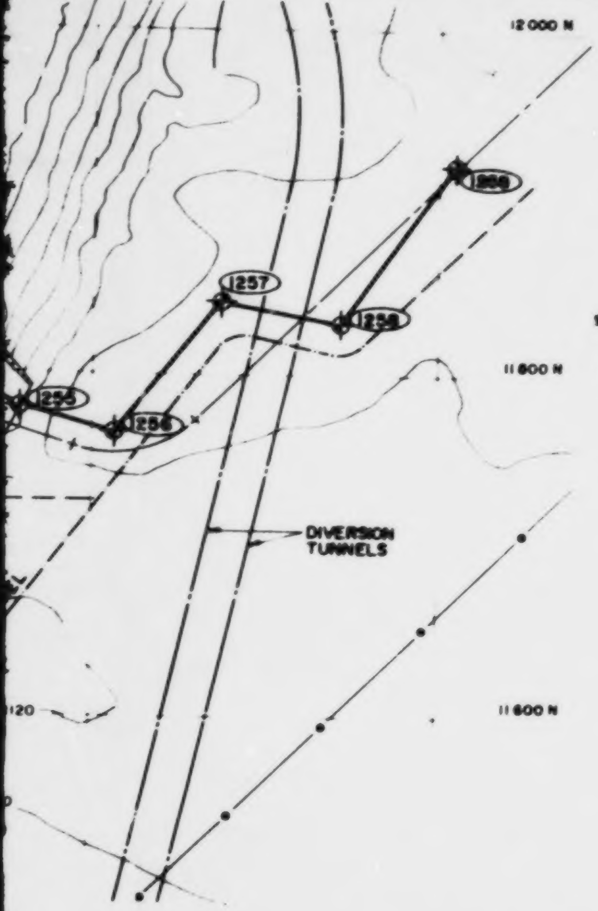
PLAN

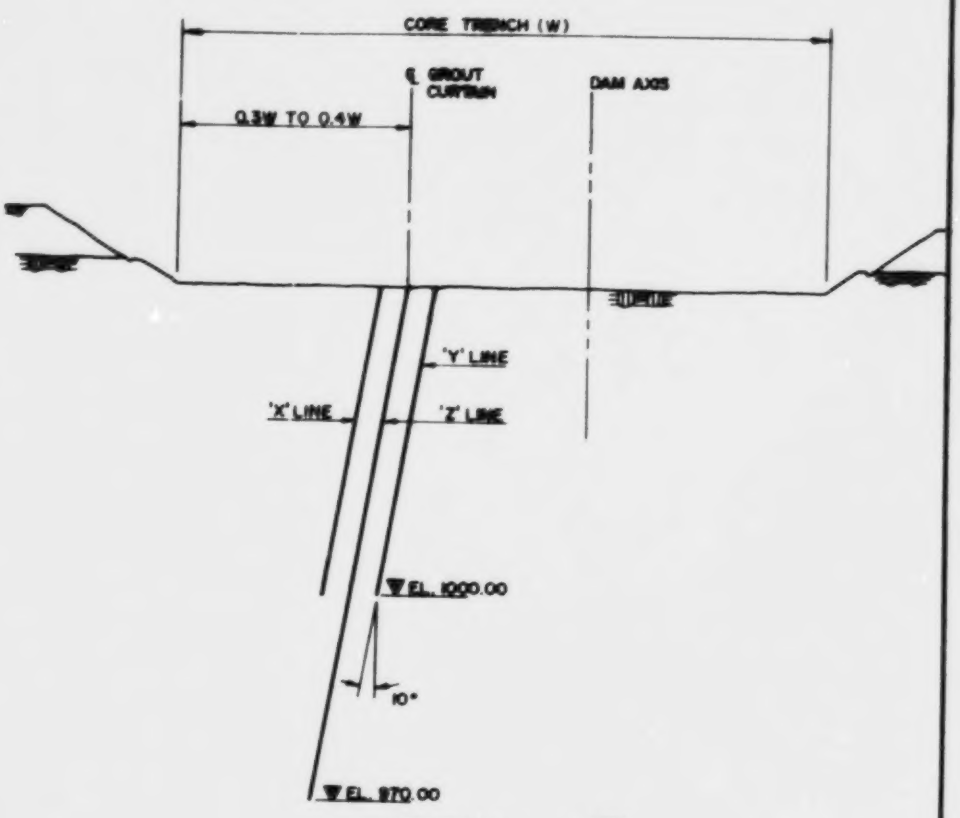

1250
1251
1252
1253
EXCAVATED SURFACE (APPROX.)
℄ SPILLWAY
40.0
EXISTING GROUND
20.0
65.0
66.0
HOL
DIRECT
TO B
CHANG
BEDROCK SURFACE (APPROXIMATE)
30° (TYP.)
EL. 1035.0
BOTTOM OF P HOLES IN LINES 'X' & 'Y'.
BOTTOM OF E HOLES IN LINE 'Z'.
EL. 970.0
BOTTOM OF E HOLES IN LINE 'Z'.
BOTTO
IN LI

GROUT CURTA

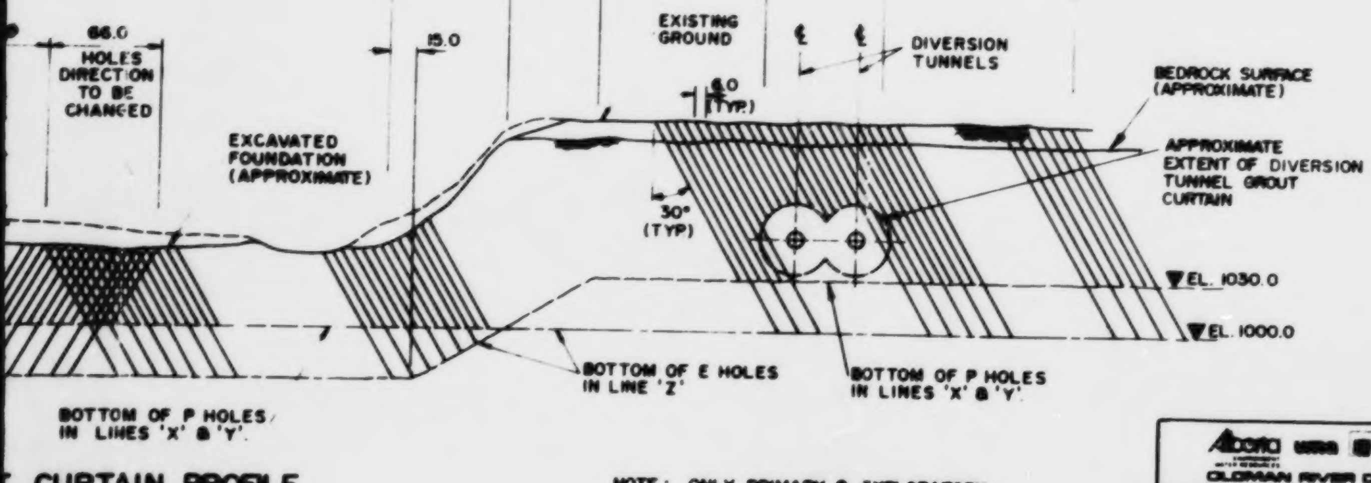
LOOKING UPSTREA

0 40
40 20


12 000 N
1259
1257
1258
1255
1256
11 800 N
DIVERSION TUNNELS
1120
11 600 N


CORE TRENCH (W)
℄ GROUT CURTAIN
DAM AXIS
0.3W TO 0.4W
'Y' LINE
'X' LINE
'Z' LINE
EL. 1000.00
10°
EL. 970.00

SECTION (A)
N.T.S.


1254
1255
1256
1257
1258
1259
66.0
HOLES DIRECTION TO BE CHANGED
15.0
EXISTING GROUND
℄
℄
DIVERSION TUNNELS
6.0 (TYP.)
BEDROCK SURFACE (APPROXIMATE)
EXCAVATED FOUNDATION (APPROXIMATE)
APPROXIMATE EXTENT OF DIVERSION TUNNEL GROUT CURTAIN
30° (TYP)
EL. 1030.0
EL. 1000.0
BOTTOM OF E HOLES IN LINE 'Z'
BOTTOM OF P HOLES IN LINES 'X' & 'Y'
BOTTOM OF P HOLES IN LINES 'X' & 'Y'

CURTAIN PROFILE
UPSTREAM

40 80 120m

NOTE: ONLY PRIMARY & EXPLORATORY HOLES SHOWN (AS 'P' AND 'E' HOLES RESPECTIVELY)

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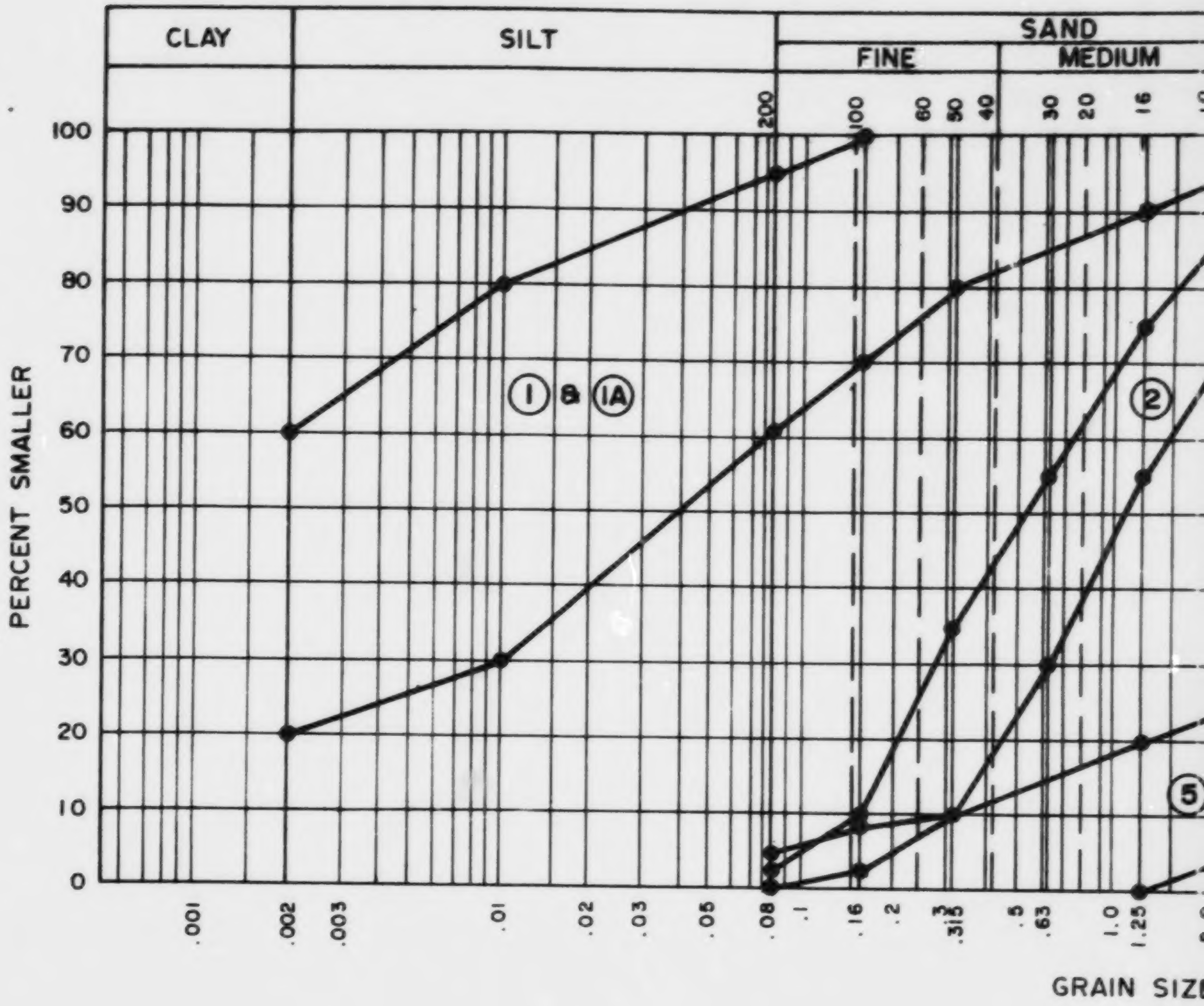
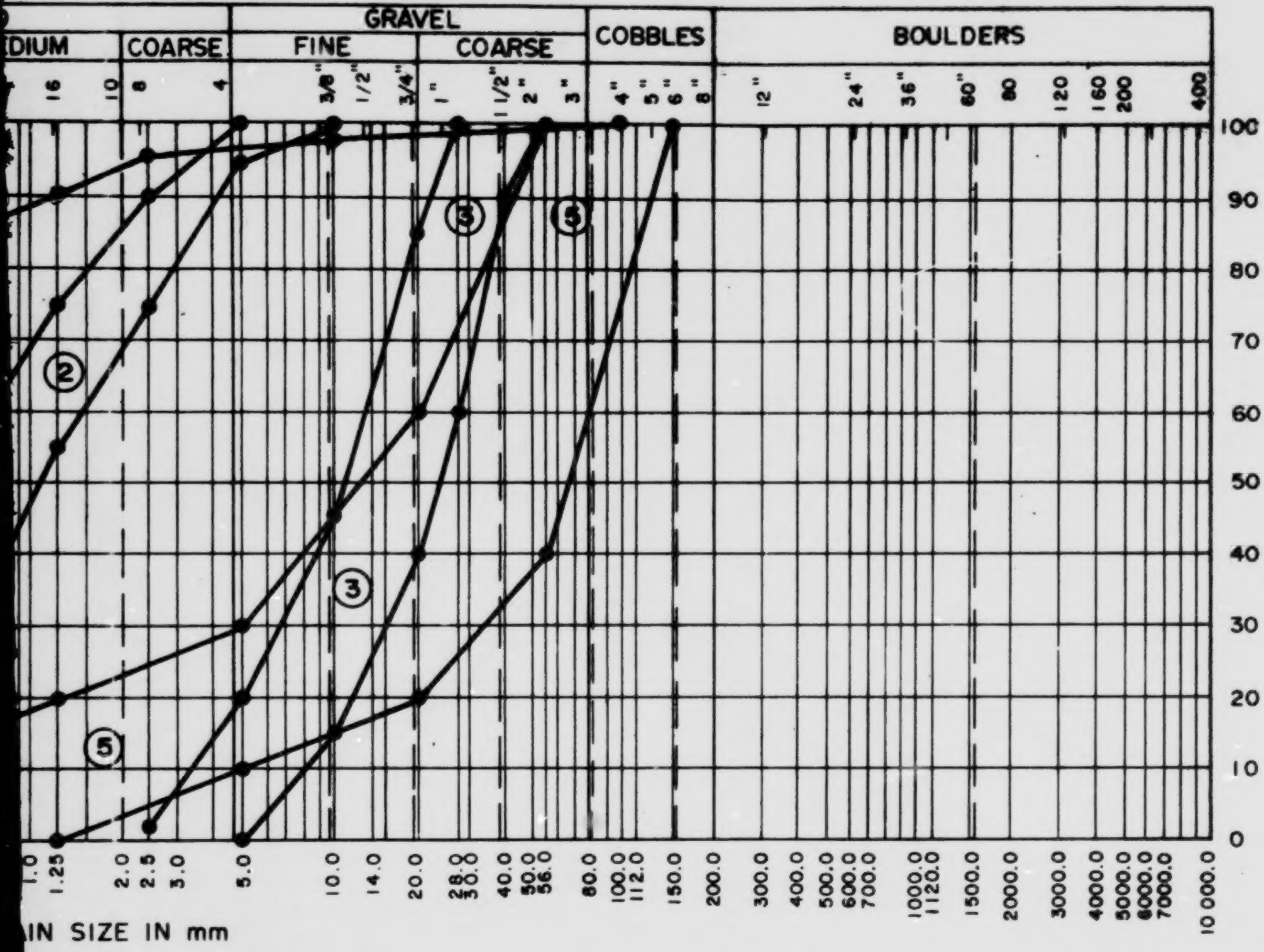
OLDMAN RIVER DAM

GROUT CURTAIN

PLATE 5


CLAY
SILT
SAND
FINE
MEDIUM
200
100
60
50
40
30
20
16
100
90
80
70
60
50
40
30
20
10
0
PERCENT SMALLER
① & ⒾⒶ
②
⑤
.001
.002
.003
.01
.02
.03
.05
.08
.1
.16
.2
.3
.315
.5
.63
1.0
1.25
GRAIN SIZ



LEGEND:

① CORE

①A UNCLASSIFIED IMPERVIOUS FILL

② FILTER

③ DRAIN

⑤ GRANULAR FILL

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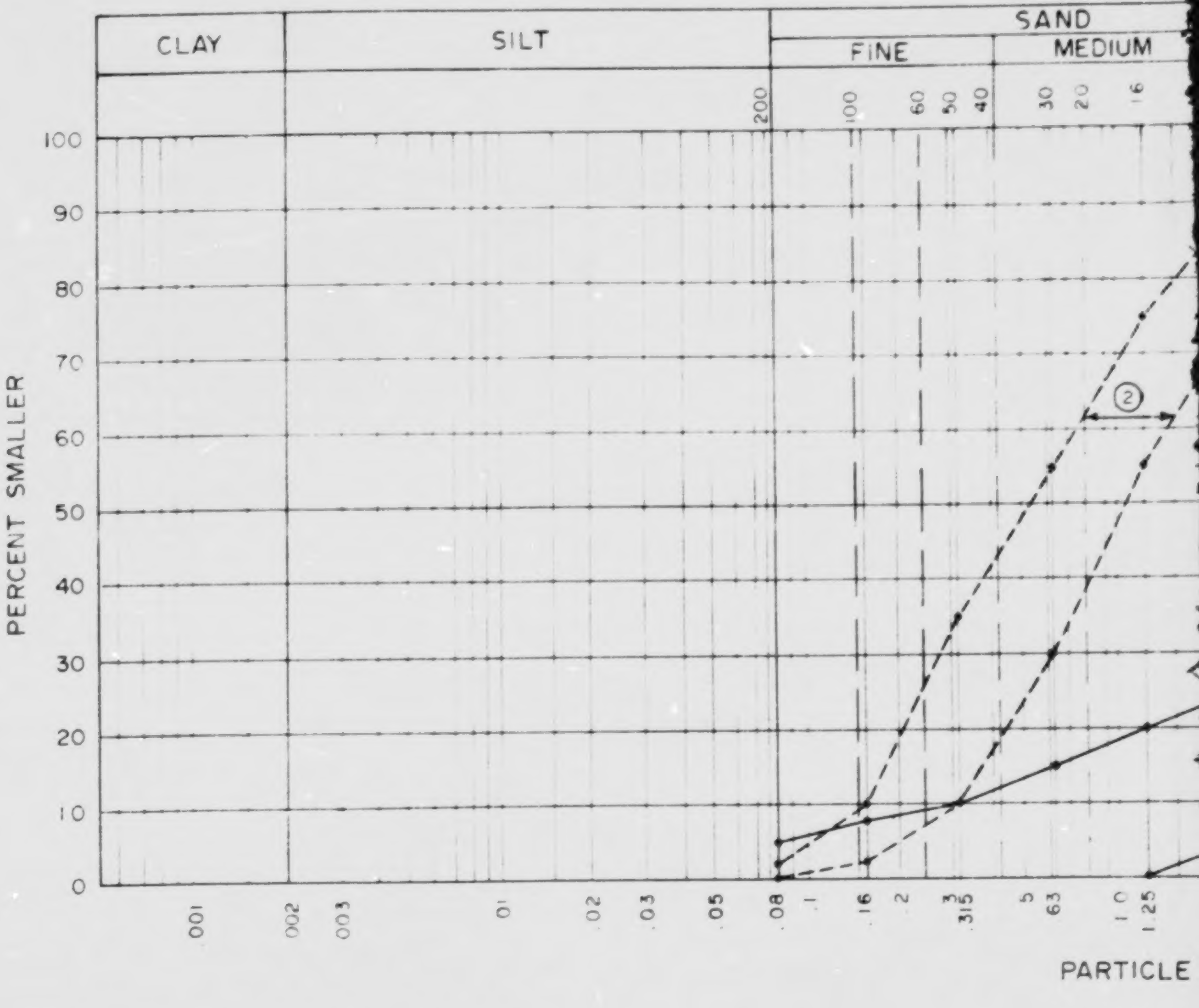
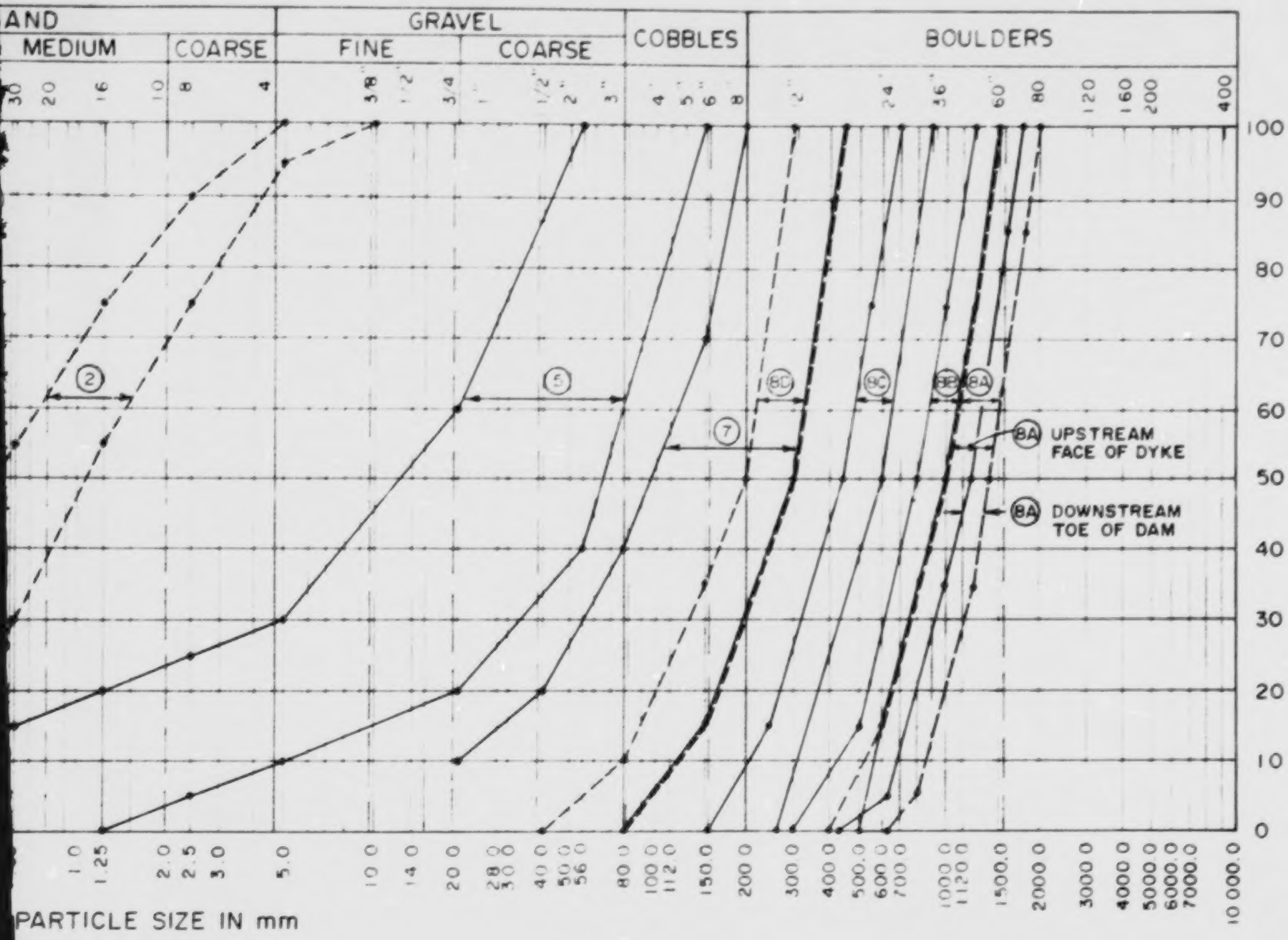
SPECIFIED GRADATION
FILL ZONES

PLATE 6



LEGEND:

② FILTER
⑤ GRANULAR FILL
⑦ RIPRAP BEDDING
(8A) RIPRAP CLASS 'A'
(8B) RIPRAP CLASS 'B'
(8C) RIPRAP CLASS 'C'
(8D) RIPRAP CLASS 'D'



NOTE:

RIPRAP GRADATION ENVELOPES ARE BASED ON A MASS TO SIZE RELATIONSHIP:

$$d = 1.15 \sqrt[3]{\frac{M}{2650}}$$

WHERE d = ROCK PARTICLE SIZE (mm)

M = REQUIRED ROCK PARTICLE MASS (kg)

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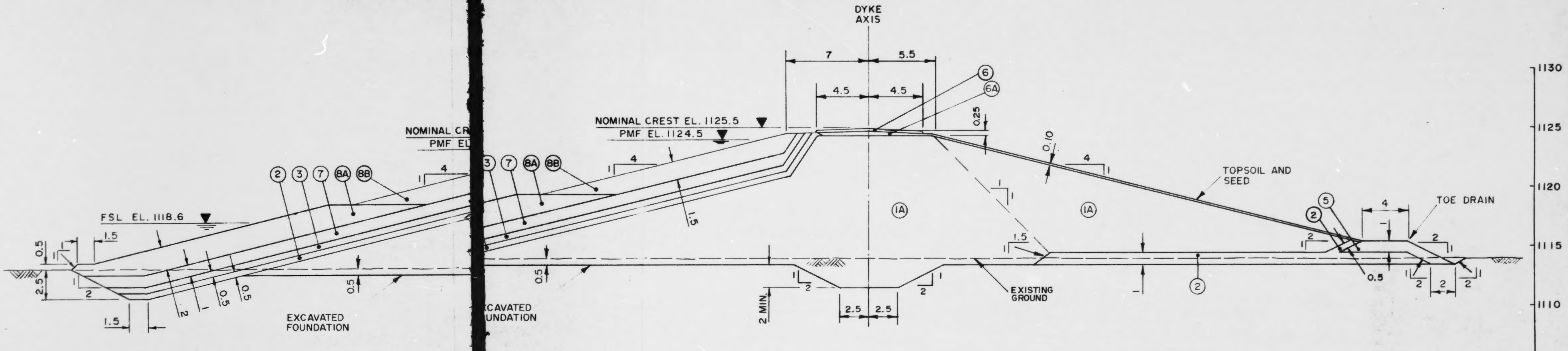
OLDMAN RIVER DAM

SPECIFIED GRADATION
RIPRAP ZONES

PLATE 7


DYKE AXIS
7
5.5
4.5
4.5
6
6A
0.25
NOMINAL CREST EL. 1125.5
PMF EL. 1124.5
NOMINAL CR
PMF EL
0.10
4
1
TOPSOIL AND SEED
1130
1125
1120
1115
1110
1105
ELEVATION (m)
2
3
7
8A
8B
FSL EL. 1118.6
1.5
0.5
2.5
2
1.5
0.5
0.5
0.5
2
1
EXCAVATED FOUNDATION
EXCAVATED FOUNDATION
1A
1A
1.5
2 MIN.
2.5
2.5
EXISTING GROUND
5
2
TOE DRAIN
0.5
2

LEGEND:

- (1A) UNCLASSIFIED IMPERVIOUS FILL
- ② FILTER
- ③ DRAIN
- ⑤ GRANULAR FILL
- ⑥ PERMANENT ROAD GRAVEL
- (6A) TEMPORARY ROAD GRAVEL
- ⑦ RIPRAP BEDDING
- (8A) RIPRAP CLASS A
- (8B) RIPRAP CLASS B

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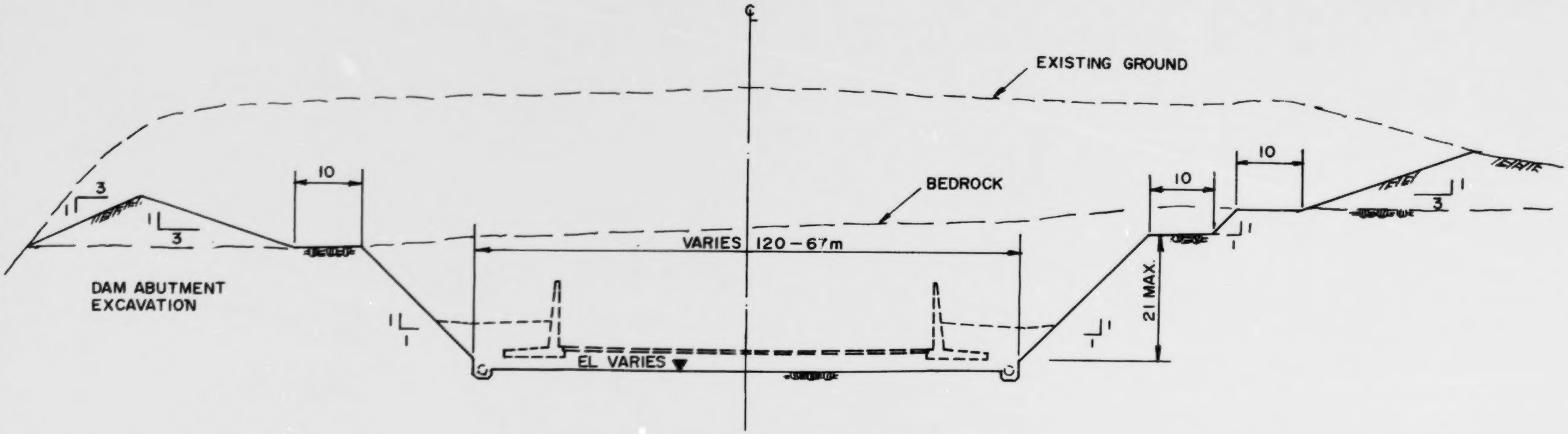
OLDMAN RIVER DAM

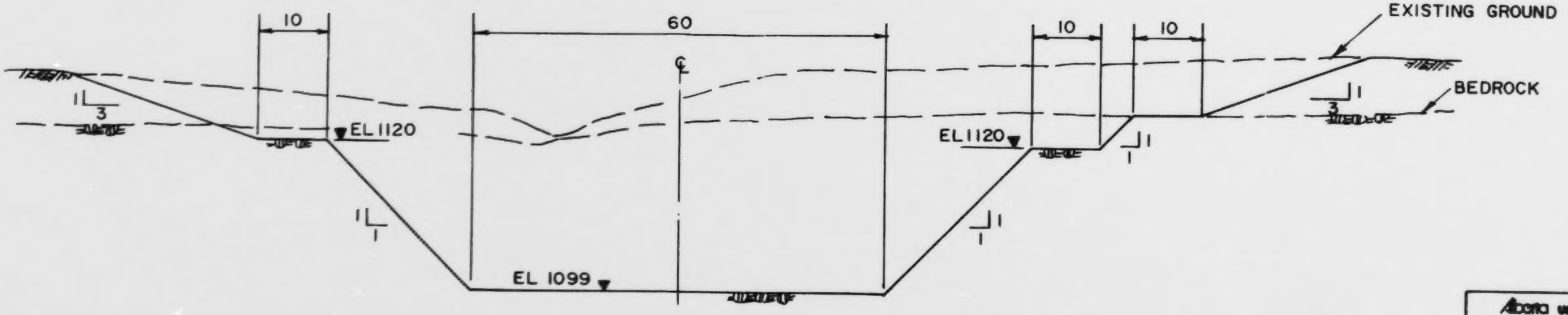
DYKE SECTION

PLATE 8


℄
EXISTING GROUND
BEDROCK
10
10
10
1
3
1
3
VARIES 120 - 67m
DAM ABUTMENT EXCAVATION
1
1
EL VARIES
1
1
21 MAX.
1
1
1
3

TYPICAL SECTION THROUGH CHUTE

N.T.S.


10
60
℄
10
10
EXISTING GROUND
BEDROCK
1
3
EL 1120
EL 1120
1
1
3
1
1
1
1
1
EL 1099

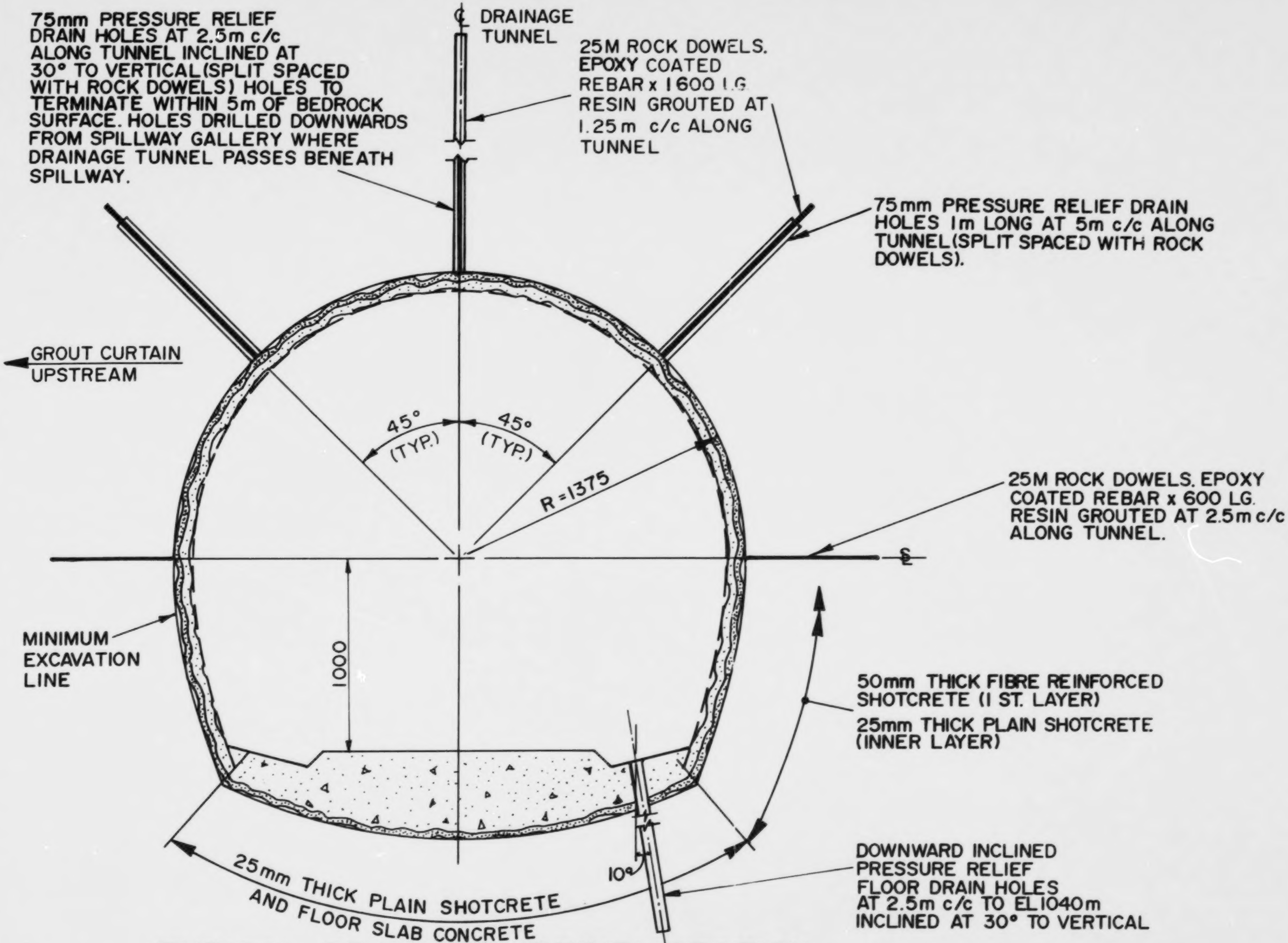
TYPICAL SECTION THROUGH APPROACH CHANNEL

N.T.S.

OLDMAN RIVER DAM

SPILLWAY EXCAVATION
SECTIONS

PLATE 9


75mm PRESSURE RELIEF DRAIN HOLES AT 2.5m c/c ALONG TUNNEL INCLINED AT 30° TO VERTICAL (SPLIT SPACED WITH ROCK DOWELS) HOLES TO TERMINATE WITHIN 5m OF BEDROCK SURFACE. HOLES DRILLED DOWNWARDS FROM SPILLWAY GALLERY WHERE DRAINAGE TUNNEL PASSES BENEATH SPILLWAY.
℄ DRAINAGE TUNNEL
25M ROCK DOWELS. EPOXY COATED REBAR x 1600 LG RESIN GROUTED AT 1.25m c/c ALONG TUNNEL
75mm PRESSURE RELIEF DRAIN HOLES 1m LONG AT 5m c/c ALONG TUNNEL (SPLIT SPACED WITH ROCK DOWELS).
GROUT CURTAIN UPSTREAM
45° (TYP.)
45° (TYP.)
R = 1375
25M ROCK DOWELS. EPOXY COATED REBAR x 600 LG. RESIN GROUTED AT 2.5m c/c ALONG TUNNEL.
℄
1000
MINIMUM EXCAVATION LINE
50mm THICK FIBRE REINFORCED SHOTCRETE (1 ST. LAYER)
25mm THICK PLAIN SHOTCRETE (INNER LAYER)
10°
25mm THICK PLAIN SHOTCRETE AND FLOOR SLAB CONCRETE
DOWNWARD INCLINED PRESSURE RELIEF FLOOR DRAIN HOLES AT 2.5m c/c TO EL 1040m INCLINED AT 30° TO VERTICAL

TYPICAL DRAINAGE TUNNEL SECTION

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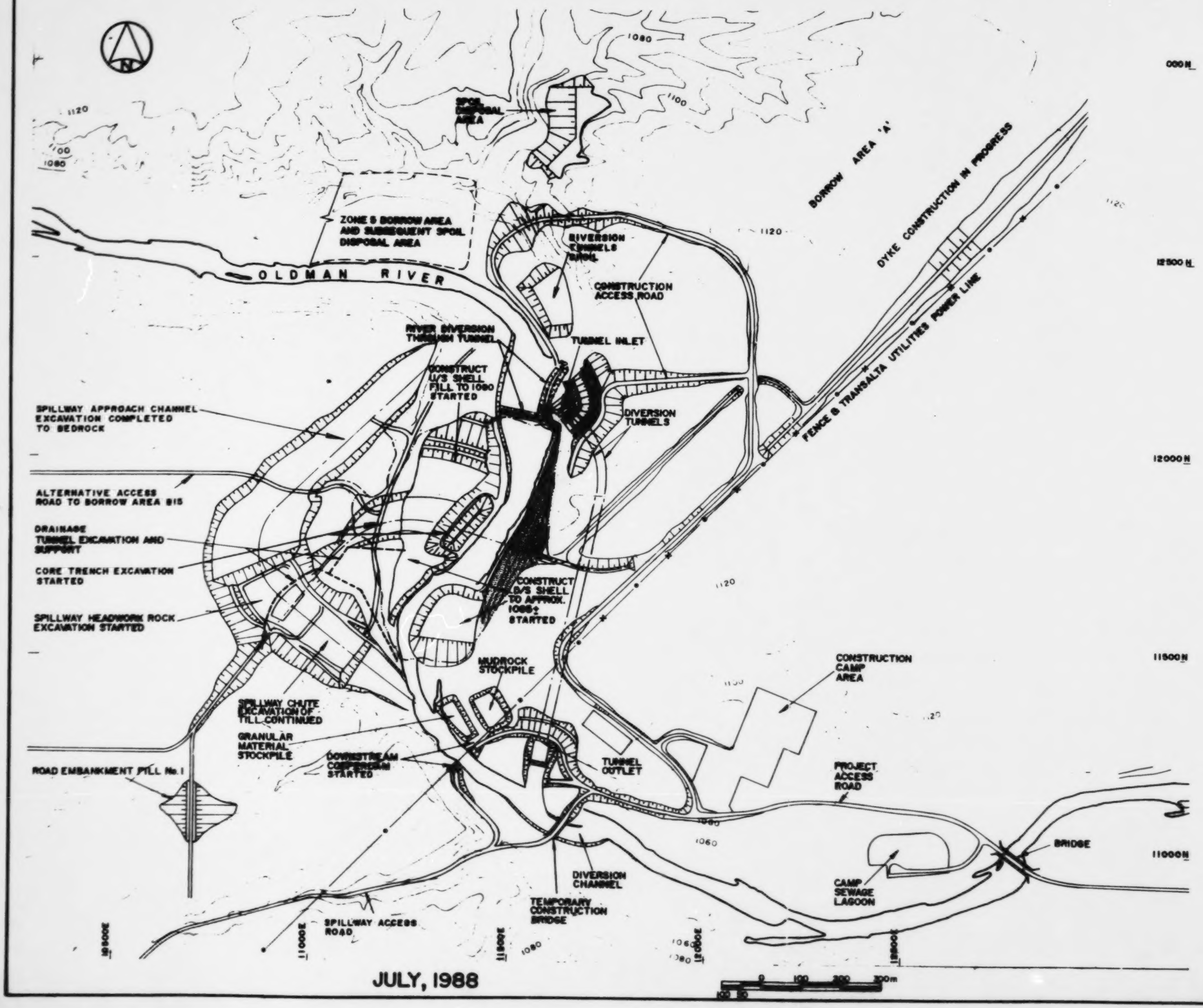
OLDMAN RIVER DAM

DRAINAGE TUNNEL
SECTION

PLATE 10

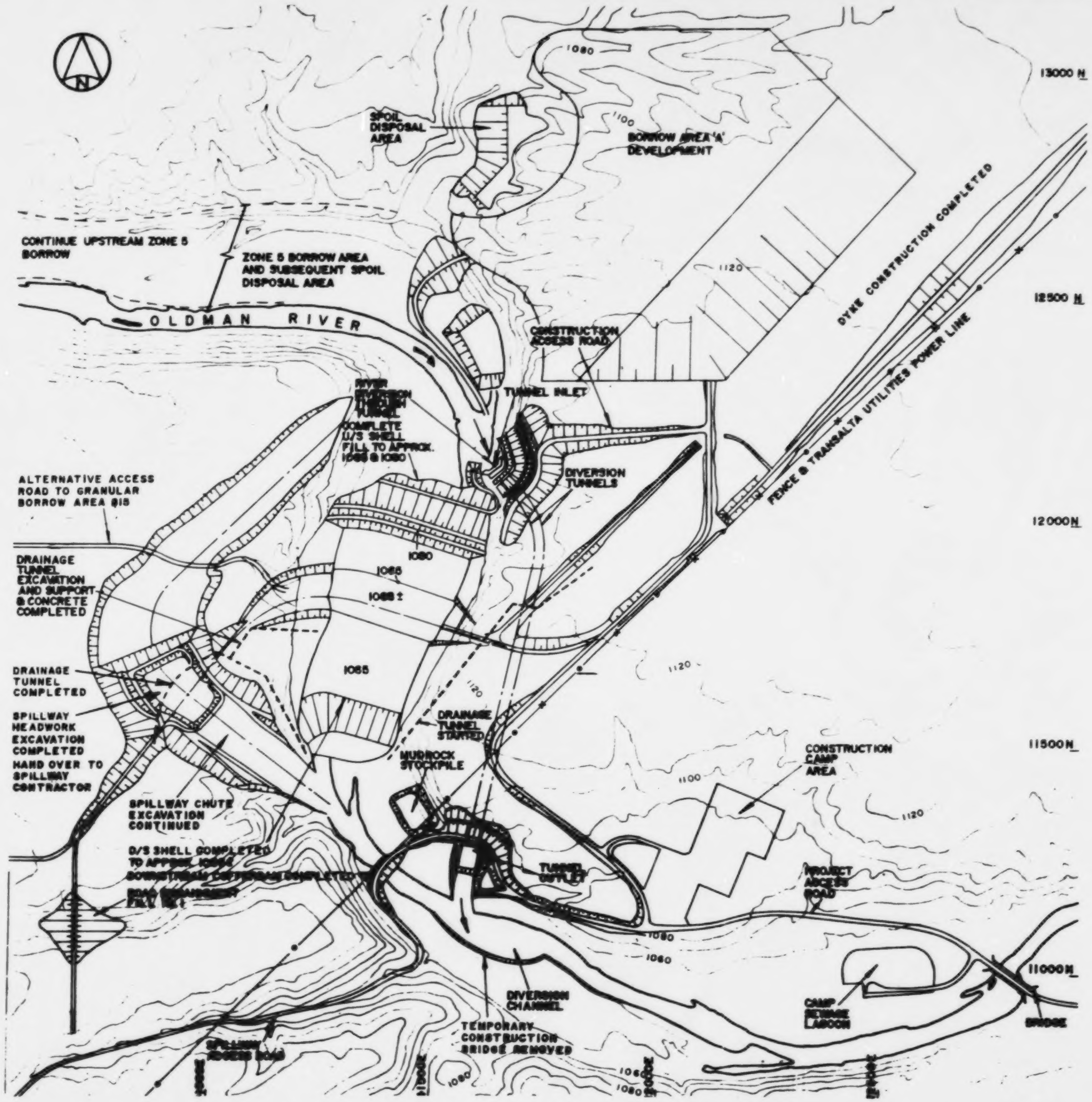

N
1120
1100
1080
1080
1100
SPOIL DISPOSAL AREA
ZONE 5 BORROW AREA AND SUBSEQUENT SPOIL DISPOSAL AREA
OLDMAN RIVER
BORROW AREA 'A'
DYKE CONSTRUCTION IN PROGRESS
1120
1120
13000N
12500N
DIVERSION TUNNELS SPOIL
CONSTRUCTION ACCESS ROAD
RIVER DIVERSION THROUGH TUNNEL
TUNNEL INLET
CONSTRUCT U/S SHELL FILL TO 1090 STARTED
DIVERSION TUNNELS
FENCE & TRANSALTA UTILITIES POWER LINE
SPILLWAY APPROACH CHANNEL EXCAVATION COMPLETED TO BEDROCK
12000N
ALTERNATIVE ACCESS ROAD TO BORROW AREA 815
DRAINAGE TUNNEL EXCAVATION AND SUPPORT
CORE TRENCH EXCAVATION STARTED
1120
CONSTRUCT D/S SHELL TO APPROX. 1085± STARTED
SPILLWAY HEADWORK ROCK EXCAVATION STARTED
MUDROCK STOCKPILE
11500N
CONSTRUCTION CAMP AREA
1100
1120
SPILLWAY CHUTE EXCAVATION OF TILL CONTINUED
GRANULAR MATERIAL STOCKPILE
DOWNSTREAM COFFERDAM STARTED
TUNNEL OUTLET
PROJECT ACCESS ROAD
ROAD EMBANKMENT FILL No. 1
1080
1060
BRIDGE
11000N
CAMP SEWAGE LAGOON
DIVERSION CHANNEL
TEMPORARY CONSTRUCTION BRIDGE
SPILLWAY ACCESS ROAD
10500E
11000E
11500E
1080
1060
1080
12000E
12500E
100 50 0 100 200 300m

JULY, 1988

OLDMAN RIVER DAM

CONSTRUCTION SEQUENCE
JULY 1988

PLATE 11

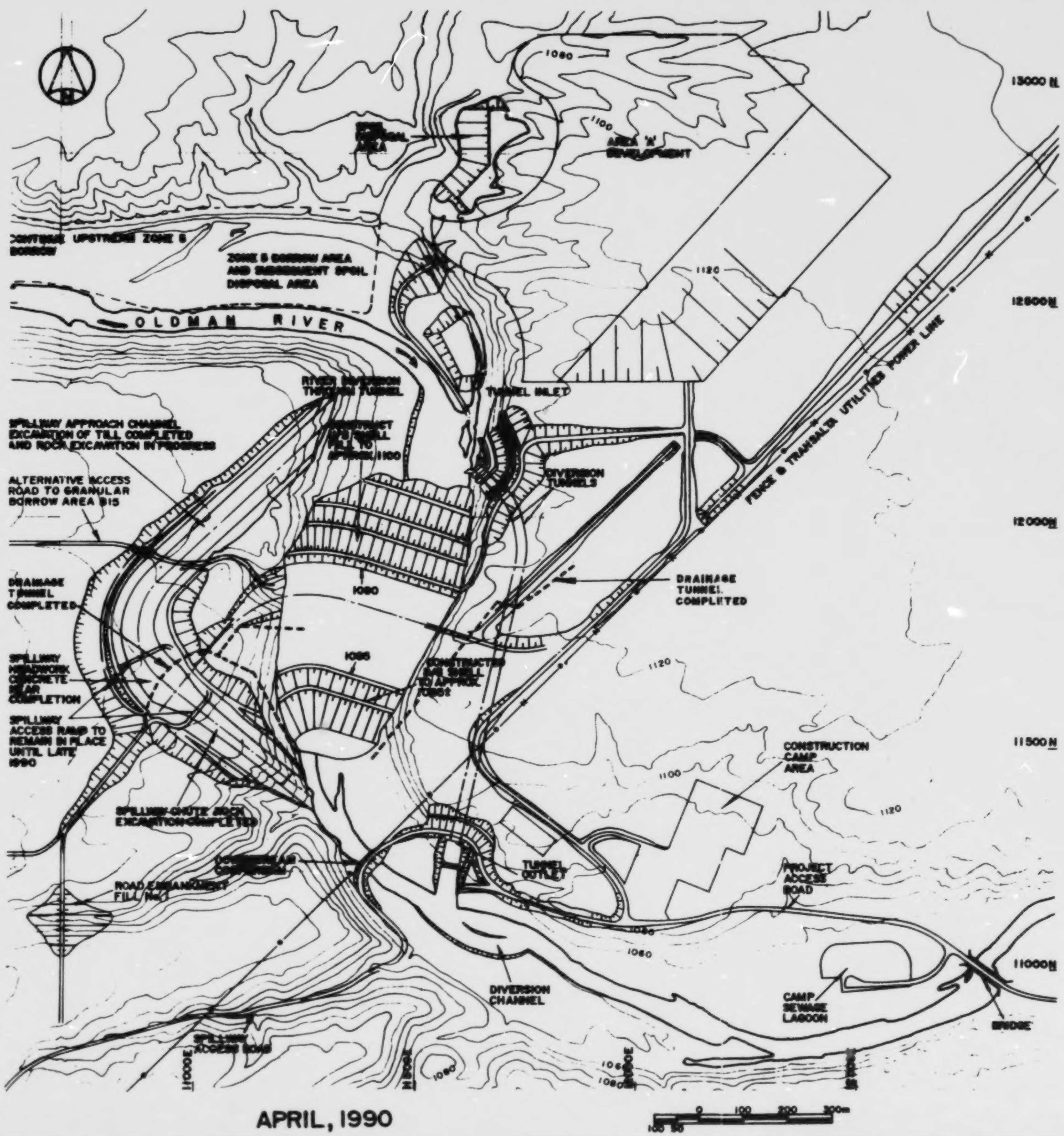

N
1080
1100
SPOIL DISPOSAL AREA
BORROW AREA 'A' DEVELOPMENT
13000 N
CONTINUE UPSTREAM ZONE 5 BORROW
ZONE 5 BORROW AREA AND SUBSEQUENT SPOIL DISPOSAL AREA
OLDMAN RIVER
1120
DYKE CONSTRUCTION COMPLETED
12500 N
CONSTRUCTION ACCESS ROAD
FENCE & TRANSALTA UTILITIES POWER LINE
RIVER DIVERSION THROUGH TUNNEL
TUNNEL INLET
COMPLETE U/S SHELL FILL TO APPROX. 1065 & 1080
DIVERSION TUNNELS
ALTERNATIVE ACCESS ROAD TO GRANULAR BORROW AREA #15
12000N
1080
1065
1068±
DRAINAGE TUNNEL EXCAVATION AND SUPPORT & CONCRETE COMPLETED
1120
DRAINAGE TUNNEL COMPLETED
1065
1120
SPILLWAY HEADWORK EXCAVATION COMPLETED
HAND OVER TO SPILLWAY CONTRACTOR
DRAINAGE TUNNEL STARTED
MUDROCK STOCKPILE
CONSTRUCTION CAMP AREA
11500N
1100
1120
SPILLWAY CHUTE EXCAVATION CONTINUED
D/S SHELL COMPLETED TO APPROX. 1065±
DOWNSTREAM COFFERDAM COMPLETED
ROAD REALIGNMENT FILL 1B±
TUNNEL OUTLET
PROJECT ACCESS ROAD
1080
1060
11000N
DIVERSION CHANNEL
CAMP SEWAGE LAGOON
BRIDGE
TEMPORARY CONSTRUCTION BRIDGE REMOVED
SPILLWAY ACCESS ROAD
1080
1060
1080
0 100 200 300m
100 50

APRIL, 1989

Alberta
OLDMAN RIVER DAM

CONSTRUCTION SEQUENCE
APRIL 1989

PLATE 12

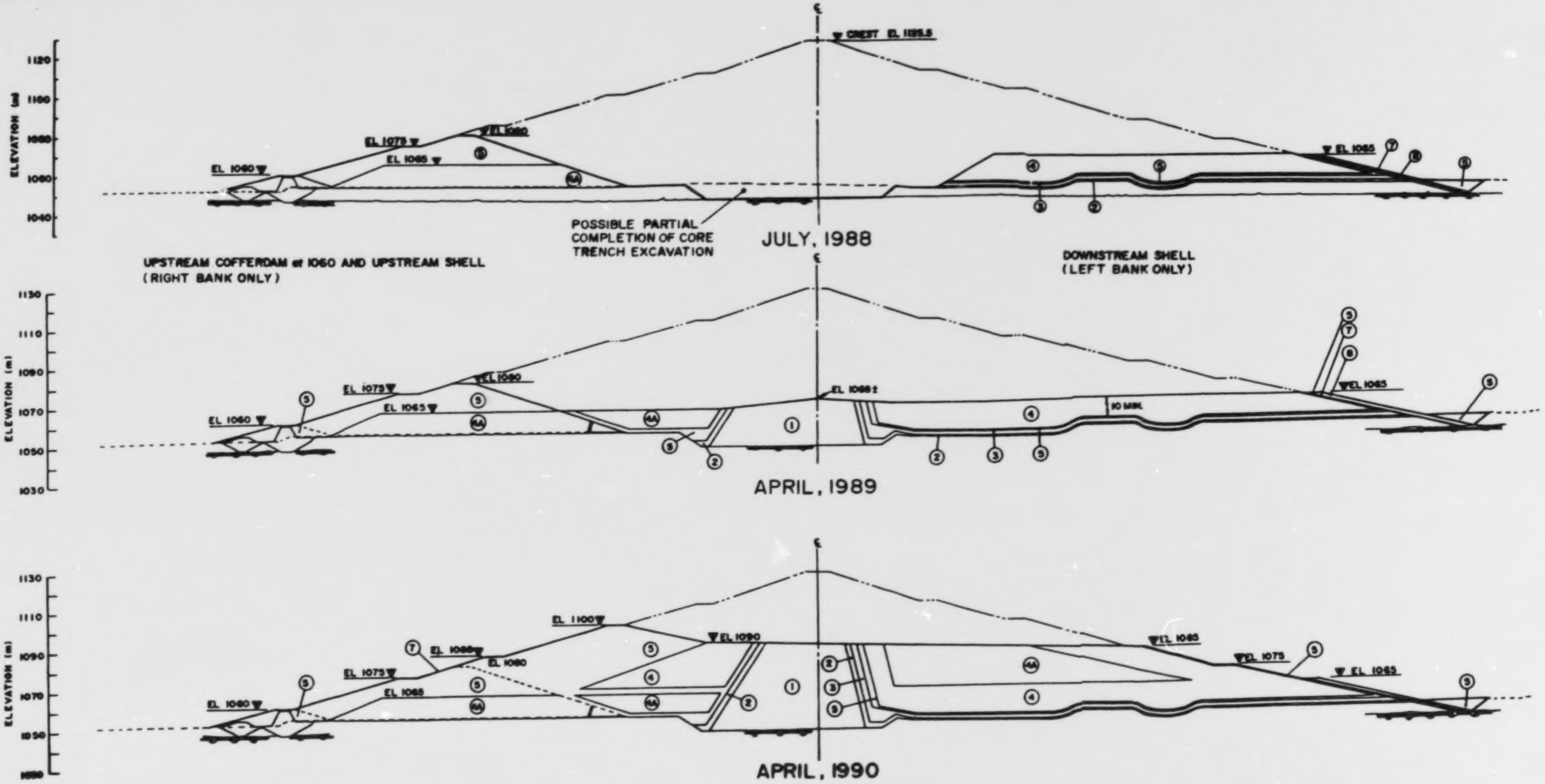

1080
1100
AREA 'A' DEVELOPMENT
13000 N
CONTINUE UPSTREAM ZONE 5 BORROW
ZONE 5 BORROW AREA AND SUBSEQUENT SPOIL DISPOSAL AREA
1120
OLDMAN RIVER
12500 N
RIVER DIVERSION THROUGH TUNNEL
TUNNEL INLET
SPILLWAY APPROACH CHANNEL EXCAVATION OF TILL COMPLETED AND ROCK EXCAVATION IN PROGRESS
ALTERNATIVE ACCESS ROAD TO GRANULAR BORROW AREA B15
DIVERSION TUNNELS
FENCE & TRANSALTA UTILITIES POWER LINE
12000 N
DRAINAGE TUNNEL COMPLETED
1080
DRAINAGE TUNNEL COMPLETED
SPILLWAY HEADWORK CONCRETE NEAR COMPLETION
1085
1120
SPILLWAY ACCESS RAMP TO REMAIN IN PLACE UNTIL LATE 1990
11500 N
CONSTRUCTION CAMP AREA
1100
1120
SPILLWAY CHUTE ROCK EXCAVATION COMPLETED
TUNNEL OUTLET
ROAD EMBANKMENT FILL No. 1
PROJECT ACCESS ROAD
1080
1060
11000 N
DIVERSION CHANNEL
CAMP SEWAGE LAGOON
BRIDGE
SPILLWAY ACCESS ROAD
1060
1080
0 100 200 300m

APRIL, 1990

Alberta
OLDMAN RIVER DAM

CONSTRUCTION SEQUENCE
APRIL 1990

PLATE 13


℄
▼ CREST EL 1125.5
ELEVATION (m)
1120
1100
1080
1060
1040
EL 1080
EL 1075
EL 1065
EL 1060
EL 1065
POSSIBLE PARTIAL COMPLETION OF CORE TRENCH EXCAVATION
UPSTREAM COFFERDAM at 1060 AND UPSTREAM SHELL (RIGHT BANK ONLY)
DOWNSTREAM SHELL (LEFT BANK ONLY)
①
②
③
④
④A
⑤
⑦
⑧
1130
1110
1090
1070
1050
1030
EL 1068±
10 MIN.
EL 1100
EL 1090
EL 1085
EL 1080
EL 1075
EL 1065

JULY, 1988

APRIL, 1989

APRIL, 1990

OLDMAN RIVER DAM

DAM CONSTRUCTION SEQUENCE

PLATE 14

OLDMAN RIVER DAM

DESIGN REPORT

APPENDIX A

CONSTRUCTION MATERIALS

SEPTEMBER, 1988

APPENDIX A

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A.1 INTRODUCTION

The inventory of naturally occurring construction materials available in the proximity of the project site was carried out as part of the preliminary design phase. The results are summarized in the Preliminary Engineering Report and are presented in detail in Appendix F of that report. During final design the most suitable deposits were selected with respect to material properties, location and volumes of usable material available. Each of the deposits were investigated further by excavating test pits, drilling Becker holes and laboratory tests. The objective of the additional field work was primarily to obtain an adequate volume of test pit and laboratory test data to allow contractors tendering the Dam and Dyke Contract to assess the quality and quantities of material available without resorting to the indirect geophysical and air photo interpretation methods used for the preliminary design work. The data presented to contractors also needed to be sufficient to determine the need for, and extent of, processing to meet the specified requirements. Plates A-1 to A-3 shows the locations of the borrow areas and the other sources of construction materials. Plate A-4 shows the specified gradations for Zone 1 Core, Zone 1A Impervious Fill, Zone 2 Filter, Zone 3 Drain and Zone 5 Granular Fill materials. This appendix updates the data analysis presented in the Preliminary Engineering Report.

A.2 TERMINOLOGY

The terms that have been used in this design report and in the contract documents are explained in this section. The "borrow deposits" were originally identified by air photo interpretation and subsequently investigated as potential sources of construction materials. References made in the Preliminary Engineering Report are to borrow deposits only.

Using the borrow deposit limits as a basis, "borrow areas" have been delineated as land beneath which suitable construction materials should be available. The limits of the borrow areas differ somewhat from those of the borrow deposits for contractual and land acquisition reasons, to allow for full exploitation of the deposits and to reflect changes to the limits to the deposit that were revealed in the later site investigation. For example the limits of borrow deposits in the river valley were restricted to those lands accessible for investigation, whereas the borrow areas encompass the material beneath the river as well as in the terraces.

A "borrow pit" is the excavation made by the contractor as he removes the material.

A.3 IMPERVIOUS FILL

Impervious fill for the dam, dyke and other embankments will be obtained from two sources, one on each side of the river.

The right bank source is the required excavation for the spillway approach channel, headworks, chute and flip bucket. Most of the overburden will be used in the permanent works. An estimated 1,400,000 m^3 of the clay till that overlies bedrock will be used as Zone 1 Core in the core of the dam. This volume will be obtained initially from the spillway excavation until all clay fill has been removed. The deficit will be made up from Borrow Area "A" on the left bank.

About 800,000 m^3 of the glaciolacustrine clay that overlies the glacial till will be used as Zone 1A Impervious Fill to construct the three road embankments to the south of the spillway. About 220,000 m^3 of the glaciolacustrine clay will be used as Zone 1A Impervious Fill to construct the 3.7 km long spillway access road from the diversion tunnels outlet area to the spillway headworks. The glaciolacustrine materials will also be acceptable in Zone 4A Unclassified Fill in the downstream shell of the dam. A further 230,000 m^3 will be required in the construction of the cofferdams. Large pockets of silt that exist in the glaciolacustrine deposit will be wasted as previous experience on the construction access road contract on the left bank has shown that the silt is dry, uniformly graded, and difficult to moisture condition and compact.

The left bank source of impervious fill is Borrow Area "A" shown on Plate A-1. In this area there is a similar sequence of glaciolacustrine silt and clay overlying glacial till. Two different glacial tills have been identified; a highly plastic clay till overlying a medium plastic silty clay till. It is estimated that about 400,000 m^3 of both types of glacial till will be used as Zone 1 Core in the core of the dam from this source.

A large volume of the glaciolacustrine silt and clay must be removed to expose the glacial till in Borrow Area "A". 230,000 m^3 of this glaciolacustrine material will be used as Zone 1A Impervious Fill to construct the dyke; some may be used in Zone 4A Unclassified Fill in the downstream shell of the dam, but the remainder must be wasted. As with the right bank, the silt component will not be used as embankment fill.

The additional site and laboratory work has essentially confirmed the properties of the overburden described in the Preliminary Engineering Report and its Appendix F. The results of all laboratory testing are summarized on Plates A-5 to A-10 inclusive, which also show the engineering properties used in the design.

A.4 ROCK FILL

The source of rock fill will be the required excavation for the Spillway approach channel, headworks, chute, flip bucket and plunge pool, the drainage tunnels, and the diversion tunnels. About 1,800,000 m^3 of the excavated rock will be used as Zone 4 Rock Fill to construct the shells of the dam. Additional rock may be placed in Zone 4A Unclassified Fill.

A.5 GRANULAR FILL

About 550,000 m^3 of Zone 2 Filter, 370,000 m^3 of Zone 3 Drain, 2,700,00 m^3 of Zone 5 Granular Fill and 140,000 m^3 of Zone 7 Riprap Bedding material for use in the construction of the dam and dyke will be obtained from the sand and gravel contained in Borrow Areas B1, B2, B7, B10, B11, B12, B13, B14 and B15 located as shown on Plates A-1 to A-3 inclusive.

All of these borrow areas except B7 and B15 are river valley deposits and will be mined partly below river level. Some washing out of fine grained particles will take place during extraction. Borrow Areas B7 and B15 are located above the present river level. Little difference has been detected between the contents of the river valley and these terrace deposits. No constraints will be imposed on the contractor concerning which borrow areas he uses or in what order they are exploited. In fact the contractor may, if he chooses, obtain part or all of his granular material from sources other than those designated.

All borrow areas have a thin mantle of top soil and silty sandy material that will be stripped and wasted to expose the granular material. The gradation requirements for Zone 5 Granular Fill have been set so that all of the contents of these deposits except the fine grained mantle can be used as embankment fill. Zone 2 Filter and Zone 3 Drain have the gradation of concrete sand and fine aggregate respectively and crushing, washing, screening, classifying and blending will be necessary to obtain granular material, meeting the specified requirements.

The Tables A-1 to A-4 inclusive and Plates A-11 to A-19 summarize the results of the laboratory tests on samples obtained from the various granular borrow areas and give an indication of the type and variation of granular material contained within them.

A.5.1 Borrow Areas B1 and B2

These are river valley alluvial deposits that are located immediately downstream of the dam and the project access bridge respectively. A summary of the investigation data is given in Table A-1 and on Plates A-11 and A-12. The contractor will be required to remove all material from Borrow Area B1 down to bedrock or to elevation 1043 m, whatever is the higher. This area will then be reclaimed using wasted soil and rock. It is planned to develop fish spawning grounds and a recreation area as part of the reclamation. Borrow Area B2 is the site of the trial granular processing program described in Section A-6. There are existing borrow pits in both borrow areas from previous contracts let to construct the construction camp access roads and other temporary construction infrastructure. The development of this borrow area will be restricted to preserve existing trees and other natural existing assets so that it can ultimately become a recreation area.

A.5.2 Borrow Areas B7E and B7W

This river terrace deposit has been included in the Dam and Dyke contracts as a contingency against the other borrow areas not yielding sufficient granular material to complete the Dam and Dyke contracts. It is the most difficult of all borrow areas to gain access to, although, being above river level it can be worked in the dry. It is located almost 9 km from the downstream toe of the dam, measured along existing roads. A summary of the investigation data is given in Table A-2 and on Plate A-13.

A.5.3 Borrow Areas B10 to B14

These borrow areas lie in the river valley immediately upstream of the dam. Borrow Area B10 together with Borrow Area B1 are the closest to the dam. Borrow Area B14, which is the most upstream area, is 7 km from the upstream toe of the dam along the river valley.

It is anticipated that most of the Zone 5 Granular Fill which has the least stringent specification, will be obtained from these borrow areas. Zone 5 Granular Fill material can be obtained pitrun and it is probable that the contractor will systematically work one side of the river, then the other, retaining an access road through the middle. This road will be removed last from the upstream end. A backhoe or possibly a dragline operation is anticipated.

The Diversion Tunnels contractor will remove some 50,000 m^3 from Borrow Area B13 for concrete aggregates prior to the area being made available to the Dam and Dyke contractor. A summary of the contents of the borrow areas is given in Table A-3 and Plates A-14 to A-18 inclusive.

A.5.4 Borrow Area B15

This borrow area comprises two terraces located south of the Bitango Bridge. Of the three terraces at this location, only the upper and middle ones contain sand and gravel. There is an existing Alberta Transportation gravel pit and a crushed gravel stockpile on the upper terrace. A summary of the contents of the borrow areas is given in Table A-4 and on Plate A-19.

A.6 TRIAL GRANULAR PROCESSING PROGRAM

A study of the gradation curves of sand and gravel sampled in all of the borrow areas indicates that crushing will be necessary to meet the specified gradation limits for Zone 2 Filter and Zone 3 Drain. The most common contractual disputes in projects of this type and size involve the question of how representative of the actual material he finds in the borrow area are the results of the site investigation and laboratory testing presented to the contractor in the tender documents. They also involve the amount, type and difficulty of processing of the actual pitrun material that is required to meet the specifications. In an attempt to alleviate such possible claims and to assist contractors in developing their tender prices a trial processing program was included in the contract to construct the spillway access road. The objective was to establish on a full scale production basis, rather than on a laboratory basis, that material meeting the required gradation limits can be obtained. The materials produced during the trial program were shown to contractors during the tendering period.

The following extract from the Spillway Access Road specifications describes the program.

"a) Description of the Work

The work shall include, but shall not necessarily be limited to:

- Setting up a production crusher, screening, washing and classifying operation at the Site.
- Loading and hauling sand and gravel from three existing sand and gravel borrow pits.
- The production of six-500 m^3 stockpiles of material that meet the specified requirements for the filter and drain zones of the Oldman River Dam.
- The continuation of the processing program under the direction of the Engineer.

b) Intent

Pervious filter and drain zones will be built into the Oldman River Dam to control the seepage of water through the dam and to prevent internal erosion of material into adjacent coarser grained zones. To function successfully the gradation of the filter and drain material must lie within narrow envelopes. The investigation of naturally occurring sand and gravel deposits in the vicinity of the dam has indicated that crushing, screening, washing, classifying and blending will be necessary to produce the required gradations with a minimum of unusable by-products. The trial aggregate processing program will allow the Engineer to optimize his design by adjusting the amount and type of processing and possibly by modifying the specified gradations to make the most efficient use of the available material. The program will also demonstrate to contractors tendering for the contract to construct the dam, the amount of effort that will be required to produce the specified materials.

2-4.14.02 Definition

Aggregate Processing Plant shall mean all excavating, hauling, crushing, screening, washing, and classifying plant and other equipment required to produce the processed materials as specified herein.

Coefficient of Uniformity shall mean the ratio of the 60 percent finer particle diameter to the 10 percent finer particle diameter, as determined from the grain size curve.

2-4.14.03 Materials

The Contractor shall obtain pitrun sand and gravel material for the trial aggregate processing program from the borrow pits B2, B3 and B15 shown on the Drawings. (Note Borrow Area B3 is reserved for the Spillway Contract under which concrete aggregates as well as Zone 2 Filter and Zone 3 Drain material will be required).

The Diversion Tunnels contractor will also be obtaining sand and gravel from borrow pit B15. The Contractor shall not interfere in any manner with the Diversion Tunnels contractor's operations and shall excavate and remove the quantity of material the Contractor requires only from the location in the borrow pit and at the times specified by the Engineer.

Zone 2 Filter and Zone 3 Drain materials shall be obtained by crushing, screening, washing, classifying and blending naturally occurring mineral sand and gravel containing no flat or elongated particles.

a) Gradation Requirements

Zone 2 Filter and Zone 3 Drain materials shall be well graded within the limits following:

Metric Sieve Size (CGSB-8-GP-2M)mm	Zone 2 Filter Percentage Passing By Weight	Zone 3 Drain Percentage Passing By Weight
60.0		100
40.0		90 - 100
20.0		40 - 85
10.0	100	15 - 45
5.0	95 - 100	0 - 20
2.5	75 - 90	0 - 1
1.25	65 - 75	
0.630	30 - 55	
0.315	10 - 35	
0.160	2 - 10	
0.080	0 - 2	

The fraction of Zone 3 Drain material that is finer than the 5 mm sieve size shall not contain more than one percent of material finer than the 0.080 mm sieve size.

The coefficient of uniformity of Zone 2 Filter material shall be not less than 4.0 and not more than 6.0. The coefficient of uniformity of Zone 3 Drain material shall not be less than 3.0 and not more than 6.0.

The Zone 2 Filter material and Zone 3 Drain material shall be considered to be well graded when the grain size curve falls fully within and is parallel to, the specified gradation limits.

b) Test Methods

The processed Zone 2 Filter and Zone 3 Drain material shall be tested in accordance with the test methods set out in Section 2-4.9.02. (Note: AASHTO to designations T27 and T11 as specified).

All aggregates being stockpiled for this work shall be tested for compliance with the specifications at a rate of not less than one test following each screen change or equipment adjustment until the product lies within the specified gradation envelope. Tests shall be at a rate of one test per hour thereafter. The tests shall be conducted by the Contractor and the results shall be submitted to the Engineer as they become available.

2-4.14.03 Execution

a) Production Phase

The Contractor shall set up the aggregate processing plant for the trial in the designated site shown on the Drawings (Note: the site is in Borrow Area B2) and shall produce and place in stockpiles 500 m^3 of Zone 2 Filter material and 500 m^3 of Zone 3 Drain material from sand and gravel from each of borrow pits B2, B3 and B15. The production phase will be completed when six stockpiles, each containing 500 m^3 of material meeting the specified gradations, are produced.

Pitrun granular materials shall be excavated and hauled from the three designated borrow pits to the site designated for the aggregate plant equipment. The designated borrow pits have been partially excavated and a face developed during previous operations. The new excavation shall proceed as an extension of the previous pit by excavating from a full face to ensure the different layers and strata exposed are well mixed. Where necessary, the overburden and rejected materials from the previous operation shall be removed and disposed of in advance of the excavation for pitrun materials. Stockpiles of sand and gravel left in and around the borrow pits from previous operations shall not be disturbed by the Contractor.

No excavation shall commence until the parts of the borrow pits to be excavated have been staked out by the Engineer and all the necessary survey measurements have been taken to determine the quantity of overburden to be excavated. Following the removal of the overburden from the area and to the depth required to recover a sufficient supply of suitable gravel, the Engineer will complete overburden measurements prior to the excavation of pitrun sand and gravel. Overburden excavation shall be performed in accordance with the specification for topsoil excavation and common excavation in Section 2.4.14.03 except that the overburden removed shall be disposed of in worked-out areas of the borrow pits.

The excavation of sand and gravel material shall be extended to such depths that all suitable sand and gravel material is recovered. In no event shall usable material be wasted. Excavations shall be continued to depths below water level if suitable material is available.

All material encountered in the pit face that is suitable for the production of Zone 2 Filter and Zone 3 Drain materials shall be used. All rock particles to a maximum size of 300 mm shall be broken down by crushing. All sand size particles shall be washed when the percentage finer than the 0.080 mm sieve size for Zone 2 Filter material exceeds the specified two percent maximum. All gravel size material shall be washed when the percentage finer than the 2.5 mm sieve size for Zone 3 Filter material exceeds the specified one percent maximum or contains plastic soils or other deleterious materials.

The existing borrow pit adjacent to the designed aggregate processing site shall be used as a settling pond for the products of the washing equipment. Sufficient materials shall be excavated from this borrow pit to produce the required quantities of Zone 2 Filter and Zone 3 Drain materials before it is used as a settling pond.

The crushing, screening, washing, classifying and blending shall be conducted such that materials in the final product are uniformly mixed throughout. The Engineer will continuously observe the processed material as it is being produced and shall take samples as frequently as considered necessary for checking the gradation of the product. The Contractor shall cooperate by obtaining the samples for the Engineer's use. This may require that suitable sampling devices be incorporated into the plant by the Contractor.

The Contractor shall stockpile the processed Zone 2 Filter and Zone 3 Drain materials from each pitrun source separately in the area shown on the Drawings. The site for each stockpile shall be cleared and levelled to provide a firm and uniform base. If necessary, the topsoil shall be stripped and stockpiled separately. No separate payment will be paid for preparation of stockpile areas other than for clearing and topsoil excavation. Stockpiles of Zone 2 Filter and Zone 3 Drain materials shall be constructed by first distributing material uniformly over the entire base, and then building upwards in successive layers not exceeding a thickness of 2 m each. Each layer shall be completed over the entire area before the subsequent layer commences. Improper use of conveyor equipment or the spilling of material over the edges of the stockpile will not be permitted. If stacking conveyors are used in place of trucks to construct the stockpile, the Contractor shall construct the stockpile in 2 m layers as specified above. A dozer-equipped tractor shall continually flatten the piles when crushing is underway. The method of stockpiling shall be such that the bases for each stockpile shall be set at least 10 m clear of any adjacent stockpile and no materials from one stockpile shall be allowed to intermix with or contaminate, the materials in an adjacent stockpile. Materials shall not be allowed to segregate.

b) Optimization Phase

Following production and stockpiling of the required quantities of Zone 2 Filter and Zone 3 Drain materials, the Contractor shall continue to operate the aggregate processing plant including the equipment used to excavate in the pits and to haul sand and gravel to the site, and shall produce additional quantities of processed materials in accordance with the Engineer's requirements. The Engineer may require the Contractor to adjust, change and modify any part of the aggregate processing plant within the normal operating limitations of each individual piece of equipment. The Engineer may also modify the specified gradation envelopes and require the Contractor to produce material meeting the modified requirements. The objective

> of this phase of the program will be to optimize the quantity and gradation of the processed product, to minimize the amount of by-products that must be wasted, and to demonstrate that the specified materials can be produced from the available sources."

The trial processing program showed that the most economic way to use the granular materials available for the Dam and Dyke Contracts will be to process the pitrun material to produce Zone 7 Riprap Bedding materials as well as the Zone 2 Filter and Zone 3 Drain materials. The by-product of the processing will be gravel size material with a gradation similar to that of the Zone 3 Drain material. For economic reasons, Zone 7 Riprap Bedding for the Dyke contract will be quarried rock supplied by the Riprap Supply contractor, as there is only a small volume of processing required to produce Zone 2 Filter and Zone 3 Drain material compared with the Zone 7 Riprap Bedding material requirements.

A.7 RIPRAP

Approximately 250,000 m^3 of riprap of all classes plus 140,000 m^3 of riprap bedding are required for protection of the upstream face of the Dam, the Dyke and some of the existing river banks from wave erosion. The riprap will also be placed on the downstream toe of the Dam to protect it from erosion during high Spillway discharges. All of the riprap will be quarried rock. Only 26,000 m^3 of the riprap bedding will be quarried rock, the remainder will come from the granular borrow areas.

Although the sandstone component of the Porcupine Hill formation found at the project site has obvious haul advantages, the durability of the sandstone is questionable. The laboratory tests show significant breakdown under freeze-thaw conditions and chemical attack. The abrasion resistance would be marginally acceptable. Reconnaissance of similar sandstone outcrops in the vicinity of the project site has indicated no better quality rock.

The criteria for acceptable riprap durability are:

TEST	METHOD	REQUIREMENTS
Abrasion Resistance	Los Angeles Machine, ASTM Designation C535	Less than 40 percent loss of weight after 500 revolutions.
Freeze-Thaw	Full Immersion ASSHTO T103 (Ledge Rock Gradation)	Less than 15 percent of particles split after 20 cycles
Soundness	Magnesium Sulphate Solution, ASTM Designation C88 (Coarse Aggregate Gradation)	Less than 10 percent loss of weight after 5 cycles
Specific Gravity	ASTM Designation C127	2.60 Minimum
Absorption	ASTM Designation C127	2% Maximum

Tests on samples from a volcanic breccia prospect 2 km to the north of Coleman and the limestone in the south quarry at Summit Lime Works indicate that these prospects will be significantly more durable than the sandstone. Limestone from the vicinity of the Frank Slide along Highway No.3 could also yeild acceptable quarried rock. However there are severe limitations in this area because of the restricted development zone around the Frank Slide historic site and because of the proximity of the towns in the vicinity.

The Riprap Supply Contractor will be responsible for selecting the quarry from which acceptable riprap of the size and properties can be obtained.

The riprap supply contract will include all work necessary to quarry and obtain the required volumes of riprap in the four classes and the riprap bedding for the Dyke contract from sources to be selected by the Contractor. This contract also includes hauling the quarried rock to the project site and stockpiling it in areas to the south of Borrow Area "A".

TABLE A-1

BORROW AREAS B1 AND B2

FIELD PROGRAM	12 Test Pits 2 Becker Holes	
QUALITY	CLASSIFICATION:	GP - GM - GW
	COBBLES:	10 to 25% (Max. 30%)
	BOULDERS:	1 to 10% (Max. 15%) to 300 mm maximum diameter
	INTERBEDS:	One pocket of SP - SM in Borrow Area B1
GROUNDWATER	DEPTH:	0.0 to 5.6 m
THICKNESS	STRIPPING:	0.0 to 1.0 m (Avg. 0.5 m)
	SAND & GRAVEL:	2.1 to 5.0 m (Avg. 3.6 m)
AREA	20 ha	

TABLE A-2

BORROW AREA B7

FIELD PROGRAM	8 Test Pits	
	No Becker Holes	
QUALITY	CLASSIFICATION:	GP - GM - GW
	COBBLES:	2 to 15% (Max. 20%)
	BOULDERS:	0 to 1% (Max. 1%) to 300 mm maximum diameter
	INTERBEDS:	SP - SW interbeds common, 0.1 to 0.3m One GC interbed at 1.2m
GROUNDWATER	DEPTH:	Greater than 5.8 m
	ELEVATION:	1071 to 1086 m (Approx.)
THICKNESS	STRIPPING:	0.3 to 1.3 m (Avg. 0.8 m)
	SAND & GRAVEL:	2.0 to 5.0 m (Avg. 3.0 m)
AREA	25 ha	

TABLE A-3

BORROW AREAS B10, B11, B12, B13 AND B14

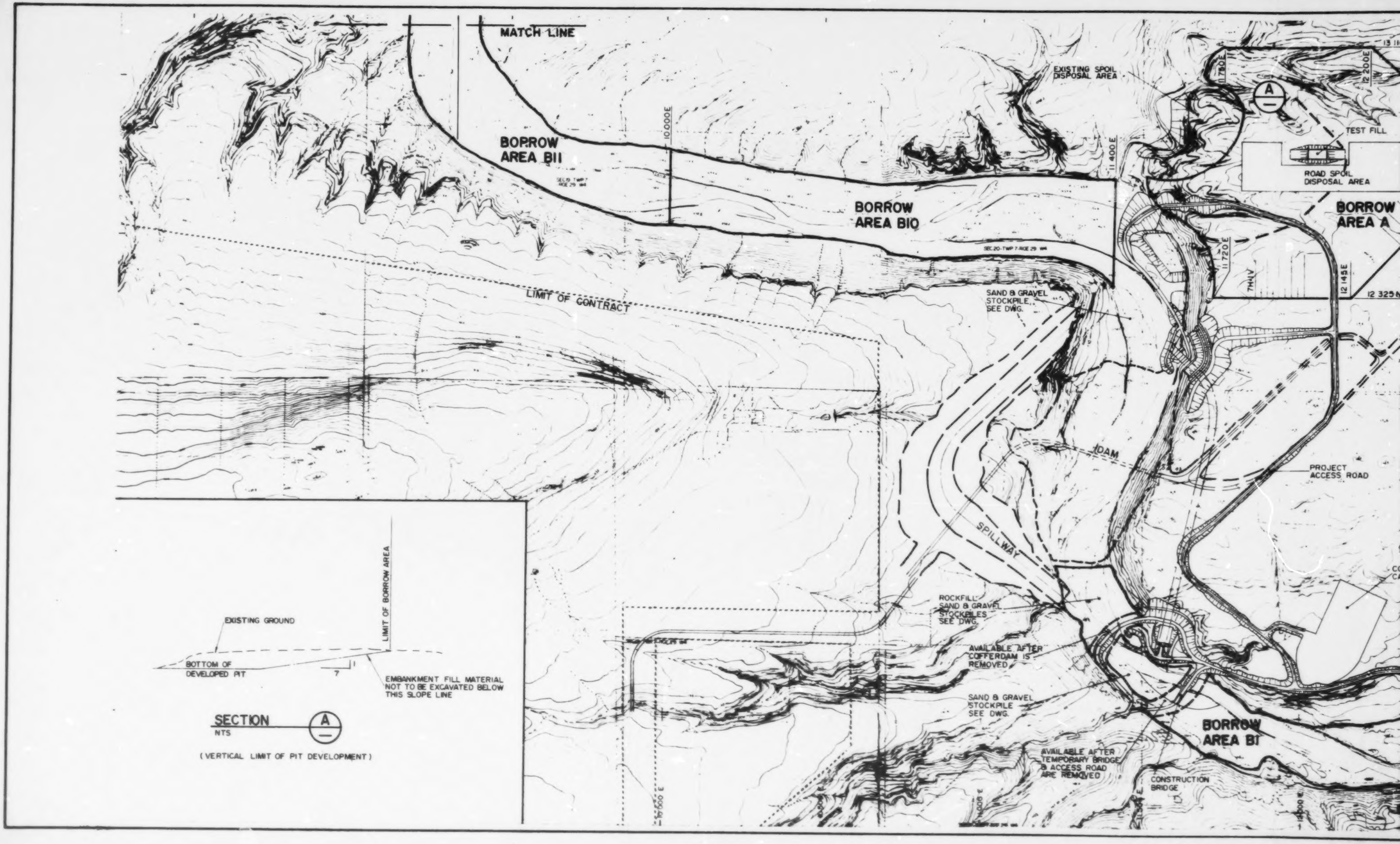
FIELD PROGRAM	53 Test Pits 5 Becker Holes	
QUALITY	CLASSIFICATION:	GW - GP, some SM and SP
	COBBLES:	10 to 30% (Max. 40%)
	BOULDERS:	1 to 15% (Max. 25%) to 500 mm maximum diameter
	INTERBEDS:	One ML bed, 0.45 m, in Borrow Area B12 One discontinuous SW - ML bed, 0.95 m in Borrow Area B14
GROUNDWATER	DEPTH:	1.5 to 5.7 m
THICKNESS	STRIPPING:	0 - 3.8 m (Avg. 0.7 m)
	SAND & GRAVEL:	2.5 - 6.4 m (Avg. 4.2 m)
AREA	360 ha	

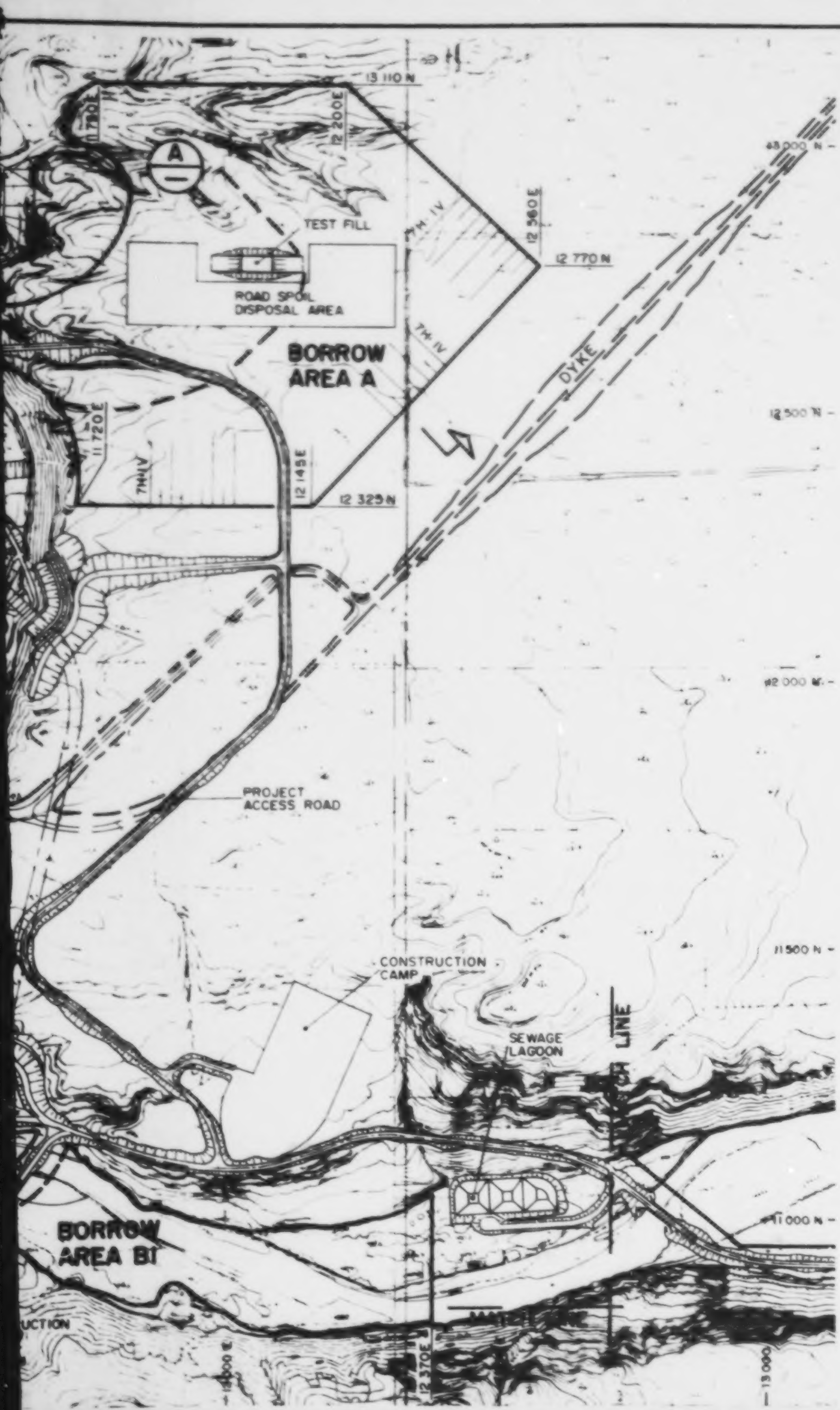
TABLE A-4

BORROW AREA B15

(Upper Deposit and Lower Deposit)

FIELD PROGRAM	24 Test Pits	
	3 Becker Holes	
QUALITY	CLASSIFICATION:GP-GM-GW	
	COBBLES:	Upper Deposit 2 to 25%
		Lower Deposit 10 to 25% (Max. 30%)
	BOULDERS:	Upper Deposit 0 to 15% (Max. 20%)
		Lower Deposit 2 to 20% (Max. 30%)
		Upper Deposit to 400 mm maximum diameter
		Lower Deposit to 750 mm maximum diameter
GROUNDWATER	DEPTH:	Deeper than 14 m
THICKNESS	STRIPPING:	Upper Deposit 0.3 to 0.9 m (Avg. 0.6 m)
		Lower Deposit 0.0 to 1.8 m (Avg. 0.8 m)
	SAND & GRAVEL:	Upper Deposit 2.4 to 4.4 m (Avg. 3.4 m)
		Lower Deposit 1.2 to 10.4 m (Avg. 5.8)
AREA	Upper Deposit 14 ha	
	Lower Deposit 42.3 ha	


MATCH LINE
BORROW AREA B11
SEC.19 TWP.7 RGE.29 WM
10,000 E
BORROW AREA B10
SEC.20-TWP.7-RGE.29 WM
11 400 E
EXISTING SPOIL DISPOSAL AREA
11 780 E
12 200 E
A
TEST FILL
ROAD SPOIL DISPOSAL AREA
BORROW AREA A
11 720 E
12 145 E
12 325 N
LIMIT OF CONTRACT
SAND & GRAVEL STOCKPILE, SEE DWG.
DAM
SPILLWAY
PROJECT ACCESS ROAD
ROCKFILL SAND & GRAVEL STOCKPILES SEE DWG.
AVAILABLE AFTER COFFERDAM IS REMOVED
SAND & GRAVEL STOCKPILE SEE DWG.
AVAILABLE AFTER TEMPORARY BRIDGE & ACCESS ROAD ARE REMOVED
CONSTRUCTION BRIDGE
BORROW AREA B1
LIMIT OF BORROW AREA
EXISTING GROUND
BOTTOM OF DEVELOPED PIT
7
1
EMBANKMENT FILL MATERIAL NOT TO BE EXCAVATED BELOW THIS SLOPE LINE
SECTION A
NTS
(VERTICAL LIMIT OF PIT DEVELOPMENT)

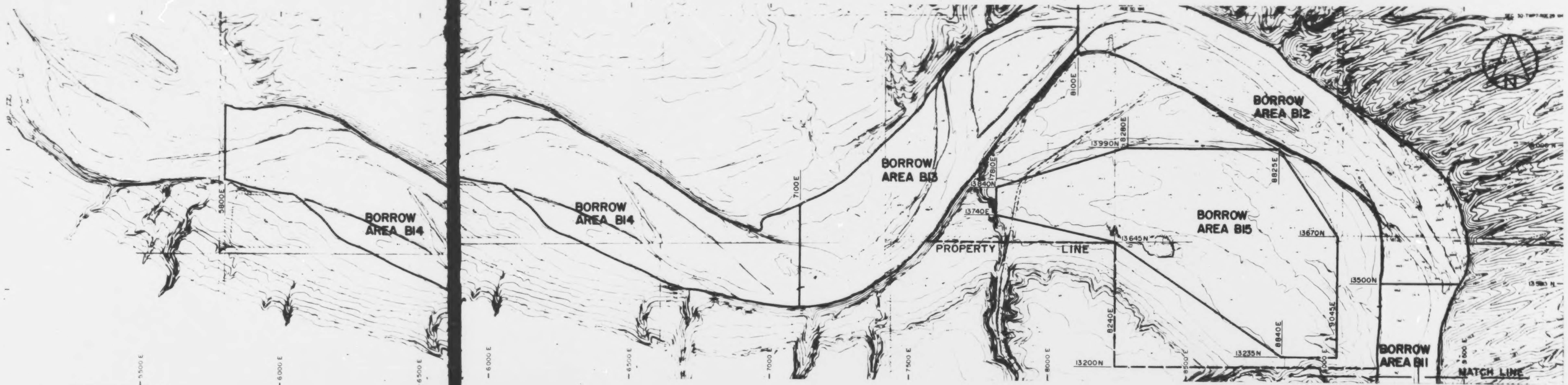

13 110 N
11 730 E
12 200 E
A
TEST FILL
ROAD SPOIL
DISPOSAL AREA
12 360 E
12 770 N
13 000 N
TH IV
BORROW
AREA A
DYKE
12 500 N
11 720 E
12 145 E
12 325 N
PROJECT
ACCESS ROAD
12 000 N
11 500 N
CONSTRUCTION
CAMP
SEWAGE
LAGOON
LINE
11 000 N
BORROW
AREA B1
UCTION
12 000 E
12 370 E
MATCH LINE
13 000

Alberta

OLDMAN RIVER DAM

APPENDIX A
BORROW AREA
LOCATION PLAN - SH 1

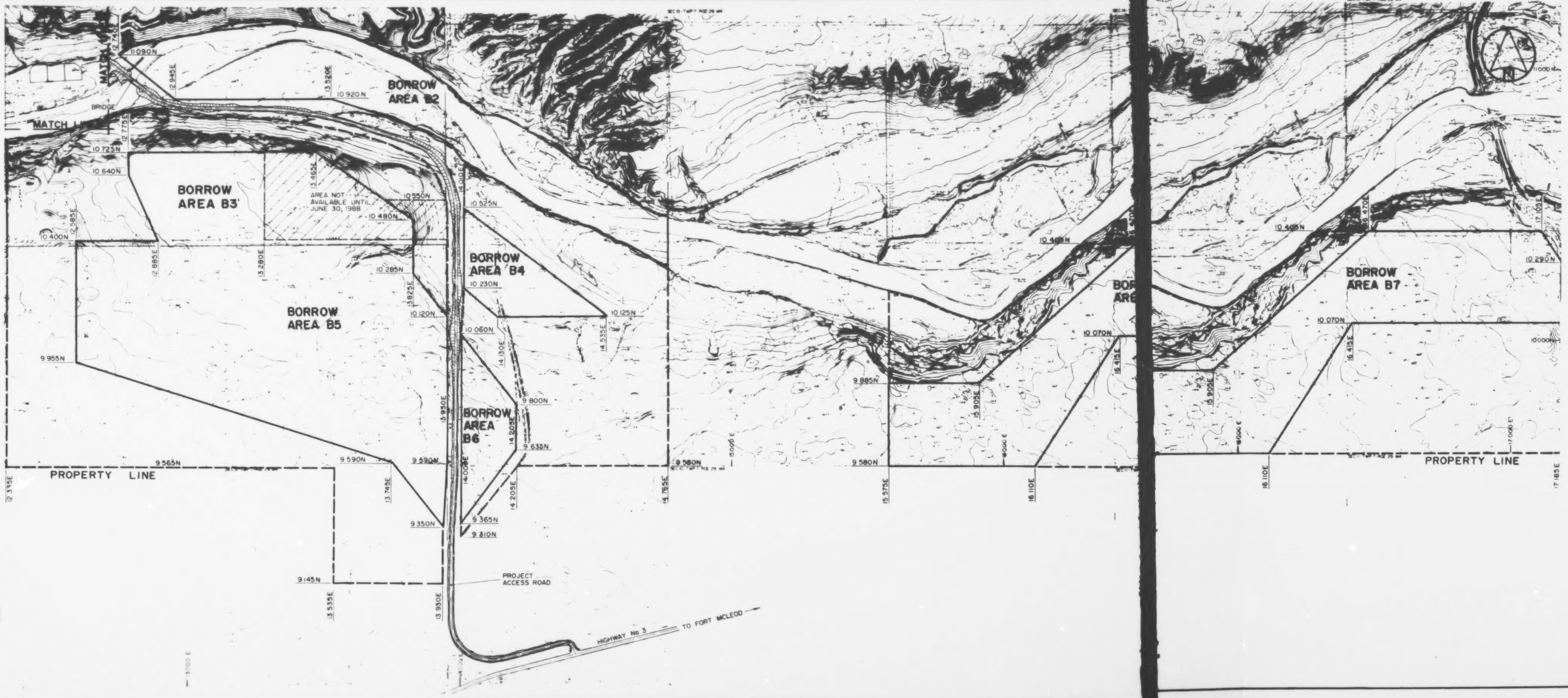
PLATE A-1


BORROW AREA B14
BORROW AREA B14
BORROW AREA B13
BORROW AREA B12
BORROW AREA B15
BORROW AREA B11
PROPERTY LINE
MATCH LINE
N
5800E
7100E
7810E
8100E
8280E
8825E
8240E
8840E
9045E
13990N
13840N
13740E
13645N
13670N
13500N
13235N
13200N
5500 E
6000 E
6500 E
6000 E
6500 E
7000
7500 E
8000 E
8500 E

Alberta
OLDMAN RIVER DAM

APPENDIX A
BORROW AREA
LOCATION PLAN - SH 2

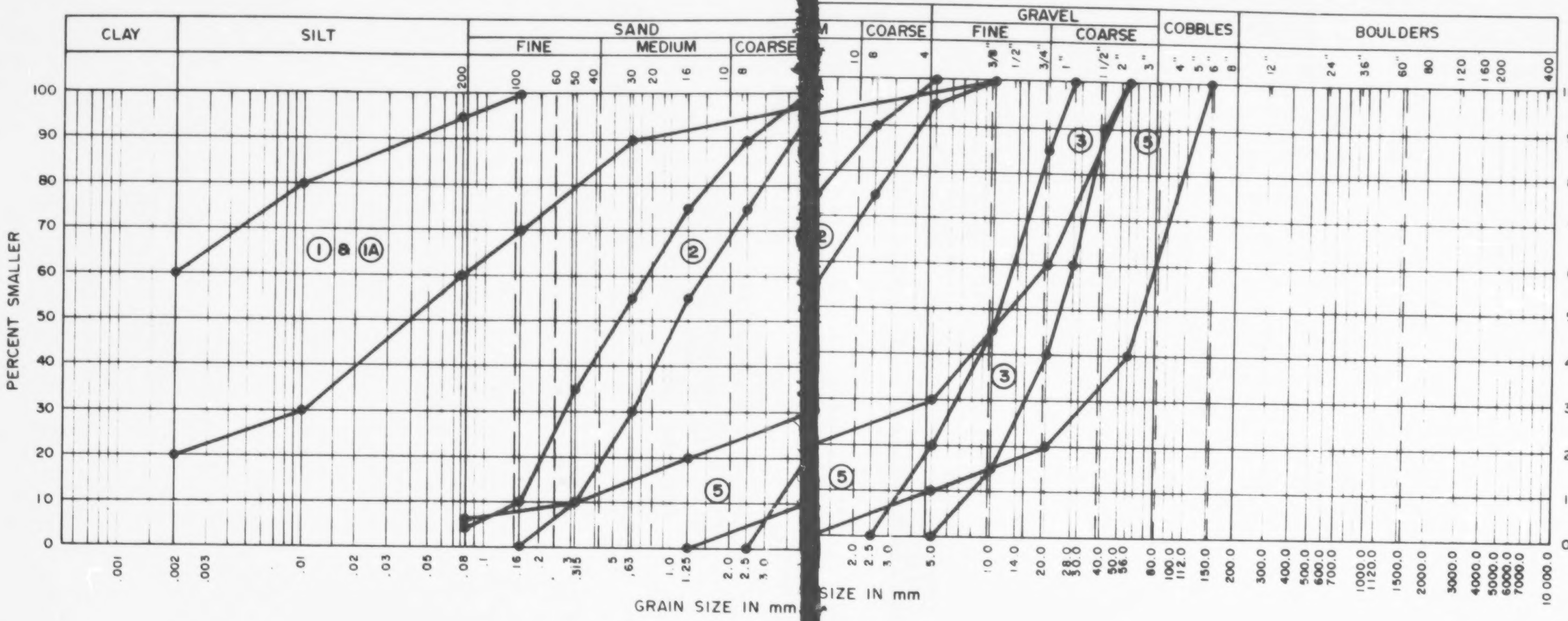
PLATE A-2


MATCH LINE
12 740E
11090N
12 945E
13 520E
10 920N
BORROW AREA B2
BRIDGE
MATCH LINE
12 775E
10 725N
10 640N
BORROW AREA B3'
13 465E
AREA NOT AVAILABLE UNTIL JUNE 30, 1988
10 550N
10 480N
10 525N
14 010E
12 585E
10 400N
12 885E
13 280E
10 285N
13825E
BORROW AREA B4
10 230N
10 125N
10 120N
BORROW AREA B5
10 060N
14 130E
14 535E
9 955N
9 800N
13 950E
BORROW AREA B6
14 205E
9 635N
9 565N
9 590N
9 590N
14 000E
9 580N
PROPERTY LINE
12 335E
13 745E
14 205E
14 765E
9 350N
9 365N
9 310N
9 145N
13 535E
13 930E
PROJECT ACCESS ROAD
HIGHWAY No 3
TO FORT MCLEOD
13000 E
15000 E
10 405N
9 885N
15 905E
9 580N
15 575E
16 000 E
16 110E
10 070N
16 415E
BORROW AREA B7
10 405N
16 470E
10 070N
16 415E
15 905E
16 000 E
16 110E
17 000 E
17 100E
10 290N
11 000N
10 000N
PROPERTY LINE
17 185E

Alberta
OLDMAN RIVER DAM

APPENDIX A
BORROW AREA
LOCATION PLAN – SH3

PLATE A -3

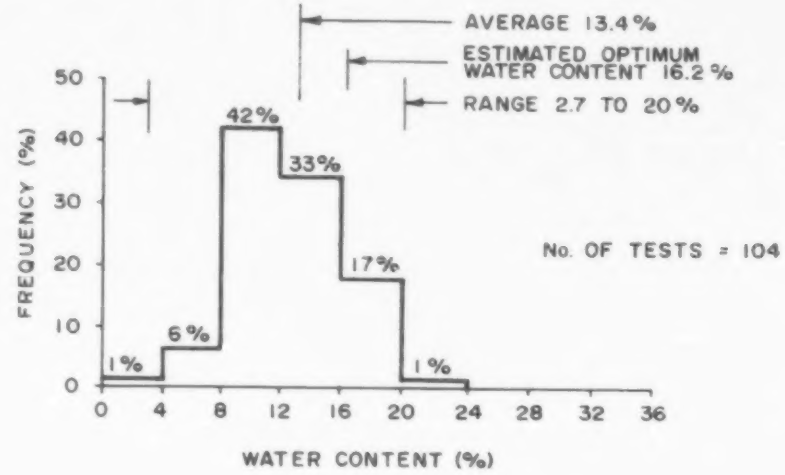


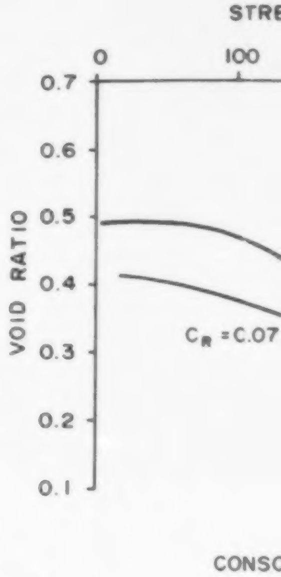
LEGEND:

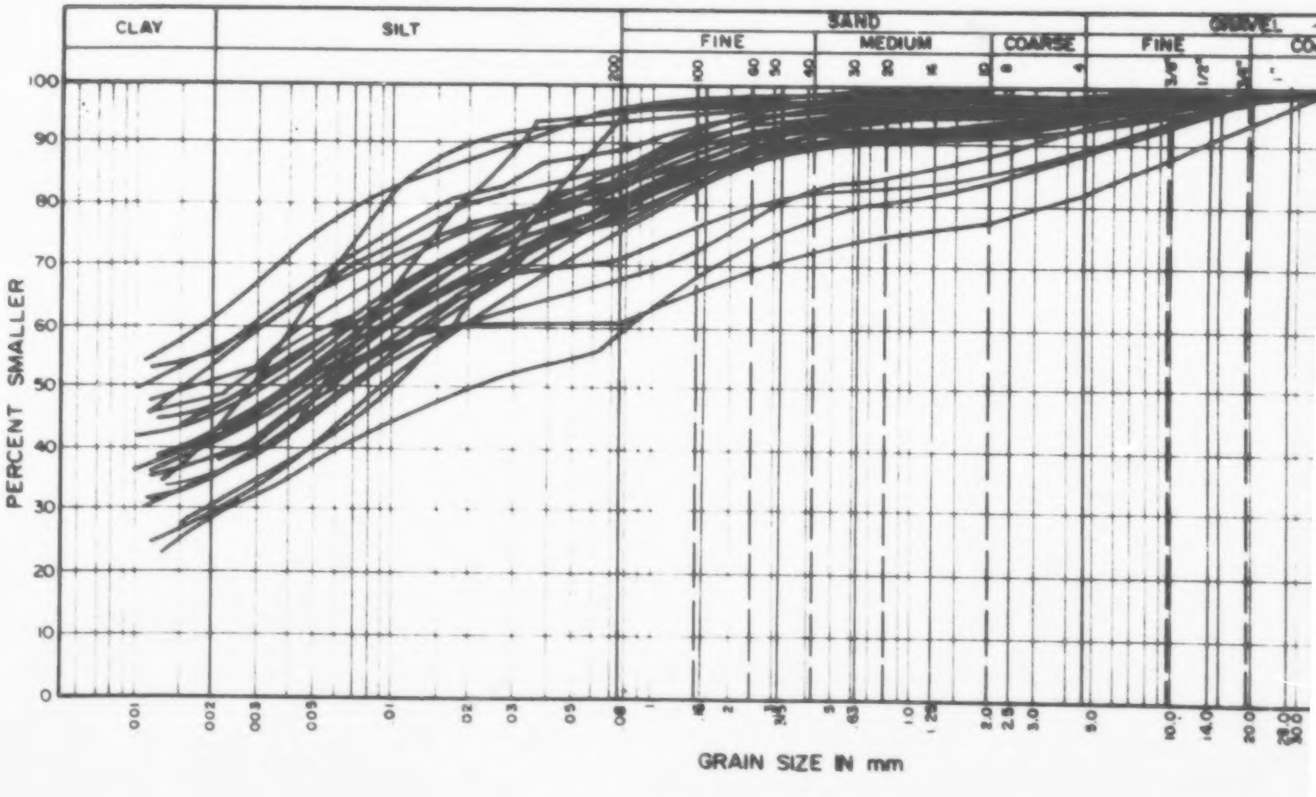
① CORE
①A UNCLASSIFIED IMPERVIOUS FILL
② FILTER
③ DRAIN
④ GRANULAR FILL

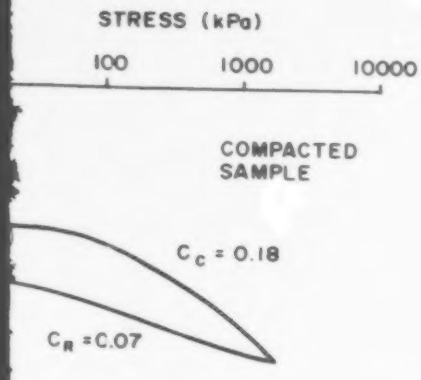
SPECIFIED GRADATION
FILL ZONES 1,1A,2,3 & 5

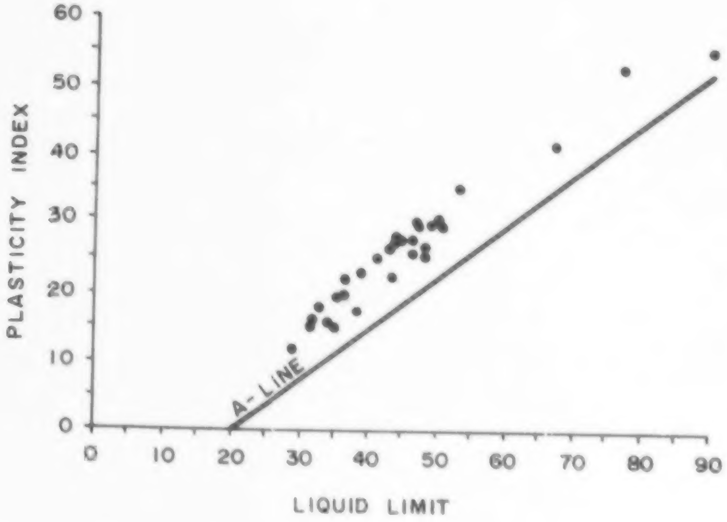
PLATE A-4


AVERAGE 13.4%
ESTIMATED OPTIMUM WATER CONTENT 16.2%
RANGE 2.7 TO 20%
FREQUENCY (%)
50
40
30
20
10
0
42%
33%
17%
6%
1%
1%
No. OF TESTS = 104
0
4
8
12
16
20
24
28
32
36
WATER CONTENT (%)

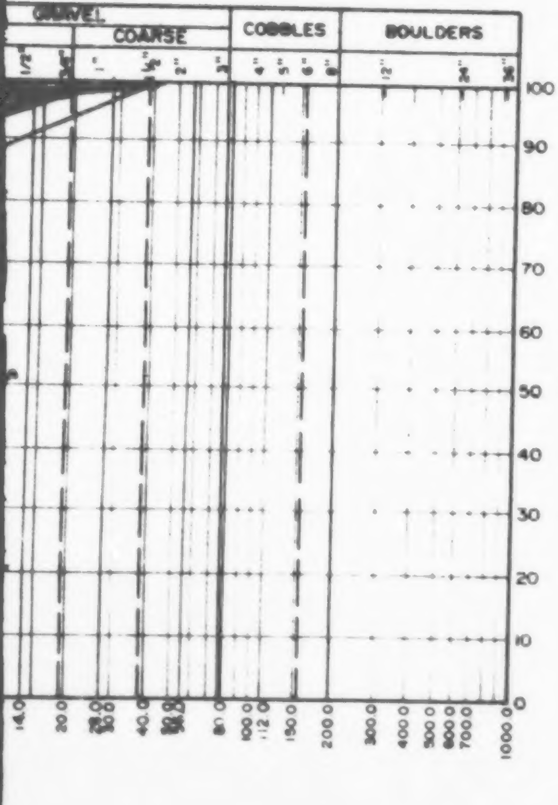

STRE
0
100
0.7
0.6
0.5
0.4
0.3
0.2
0.1
VOID RATIO
C_R = C.07
CONSC

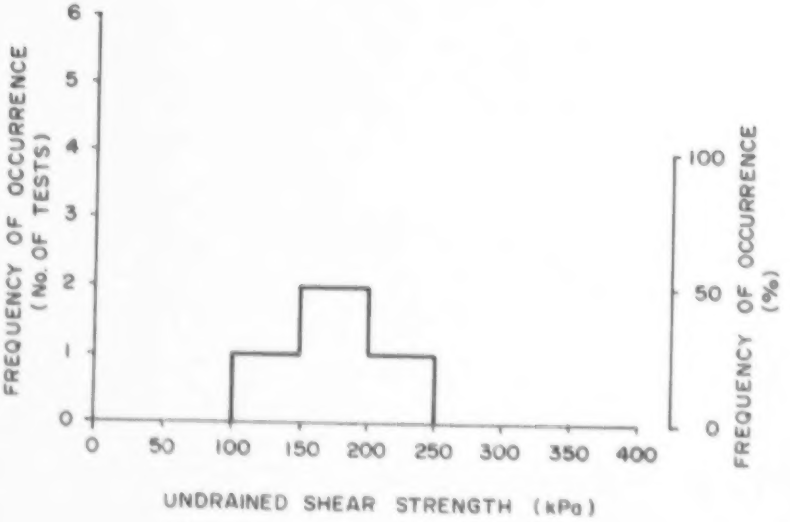

CLAY
SILT
SAND
FINE
MEDIUM
COARSE
GRAVEL
FINE
CO
200
100
60
50
40
30
20
16
10
8
4
3/8"
1/2"
3/4"
100
90
80
70
60
50
40
30
20
10
0
PERCENT SMALLER
.001
.002
.003
.005
.01
.02
.03
.05
.08
.1
.15
.2
.3
.5
.63
1.0
1.25
2.0
2.5
3.0
5.0
10.0
14.0
20.0
GRAIN SIZE IN mm





CONSOLIDATION

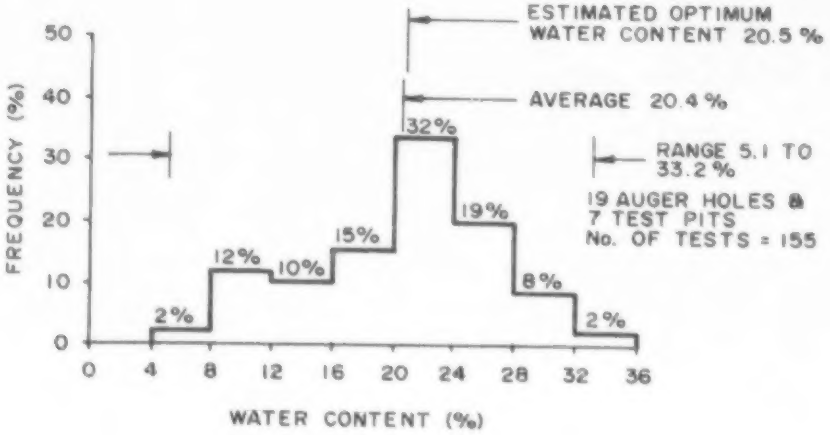


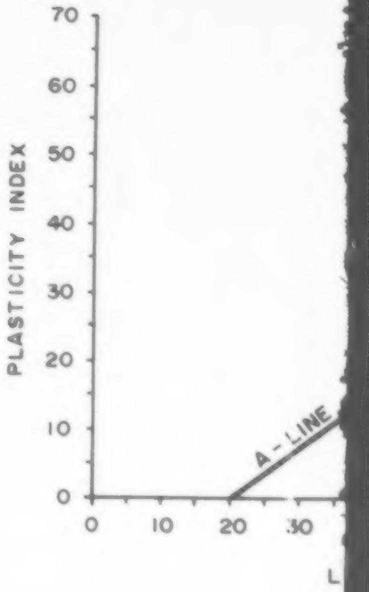


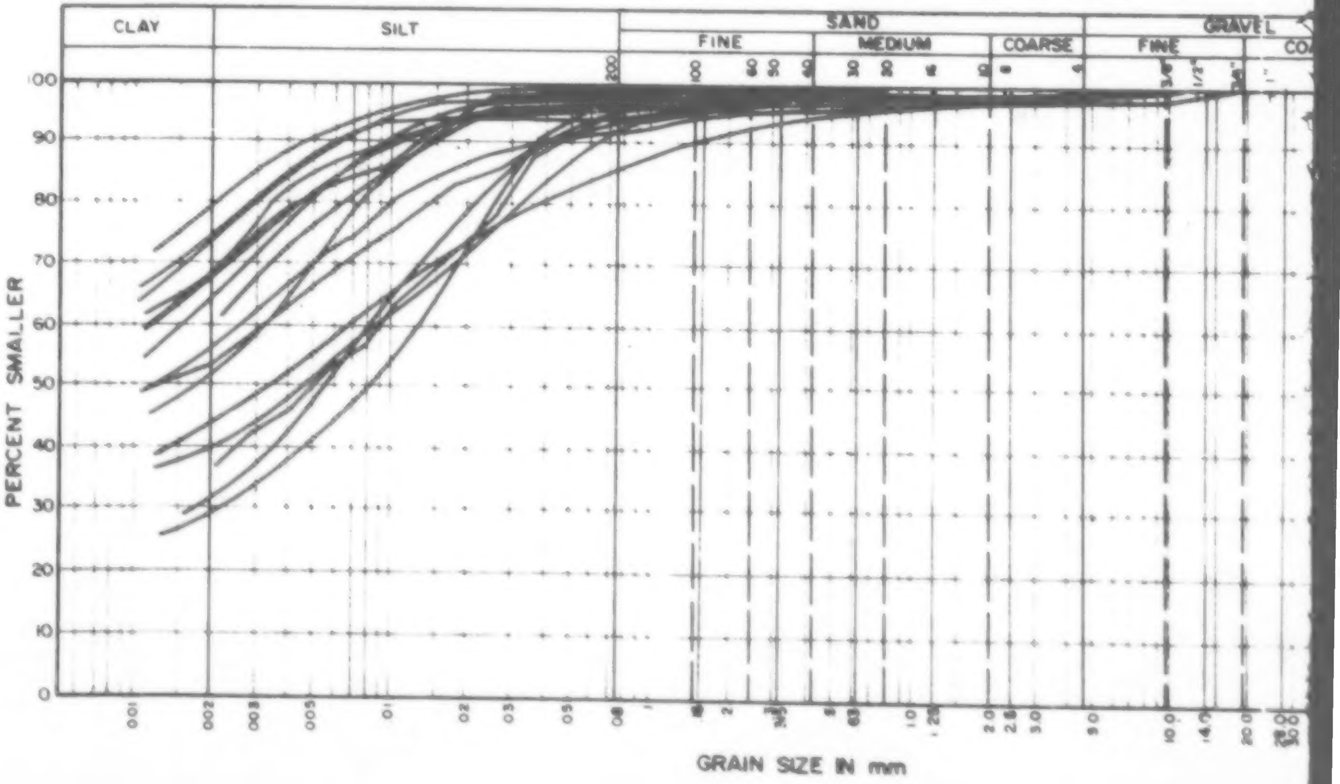
Alberta
OLDMAN RIVER DAM

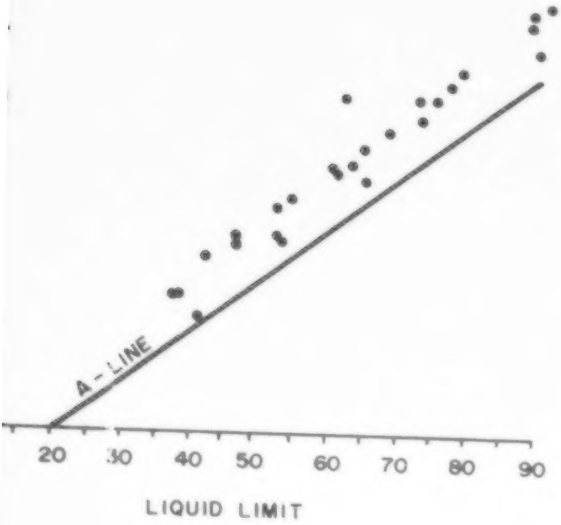
RIGHT BANK
GLACIAL TILL

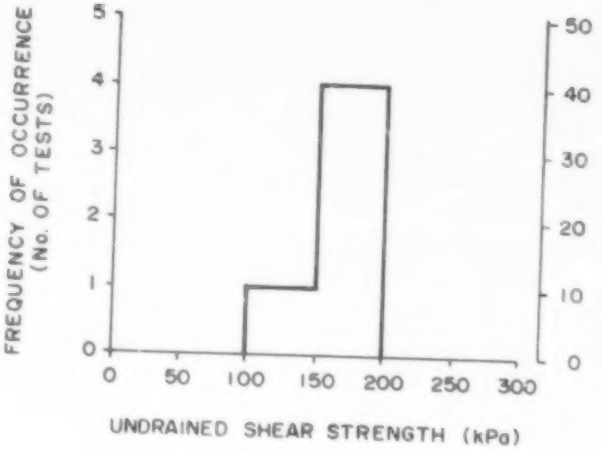
PLATE A -5

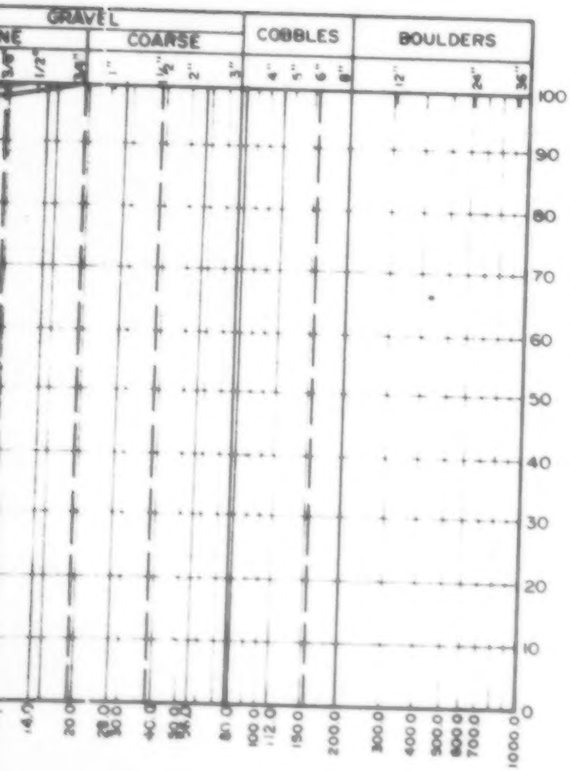

ESTIMATED OPTIMUM WATER CONTENT 20.5 %
AVERAGE 20.4 %
RANGE 5.1 TO 33.2 %
19 AUGER HOLES & 7 TEST PITS
No. OF TESTS = 155
FREQUENCY (%)
WATER CONTENT (%)
0
10
20
30
40
50
0
4
8
12
16
20
24
28
32
36
2%
12%
10%
15%
32%
19%
8%
2%


PLASTICITY INDEX
70
60
50
40
30
20
10
0
0
10
20
30
A - LINE
L


CLAY
SILT
SAND
FINE
MEDIUM
COARSE
GRAVEL
FINE
CO
200
100
3/4"
1/2"
3/8"
PERCENT SMALLER
100
90
80
70
60
50
40
30
20
10
0
GRAIN SIZE IN mm
.001
.002
.003
.005
.01
.02
.03
.05
.08
.2
.5
1.0
2.0
5.0
10.0
20.0



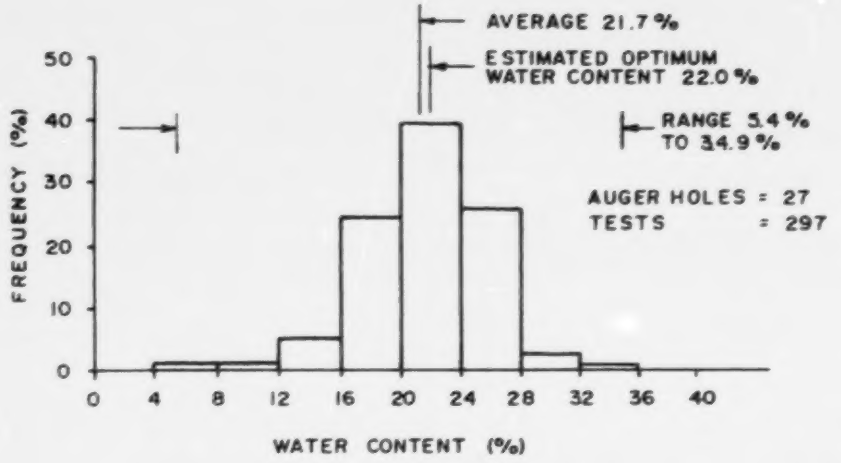




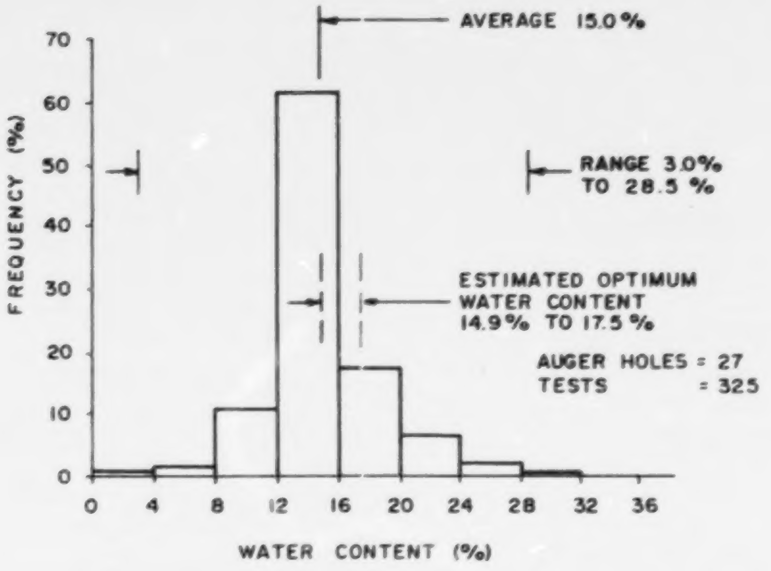
Alberta
OLDMAN RIVER DAM

RIGHT BANK
GLACIO LACUSTRINE CLAY

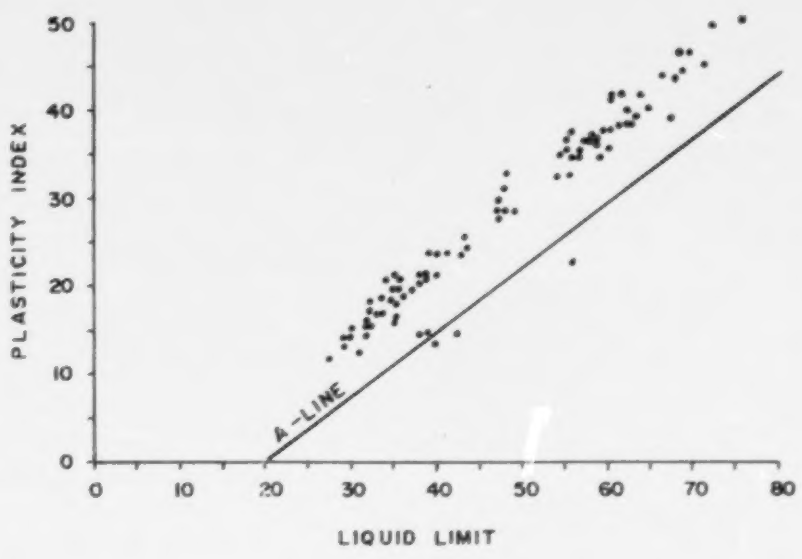
PLATE A-6



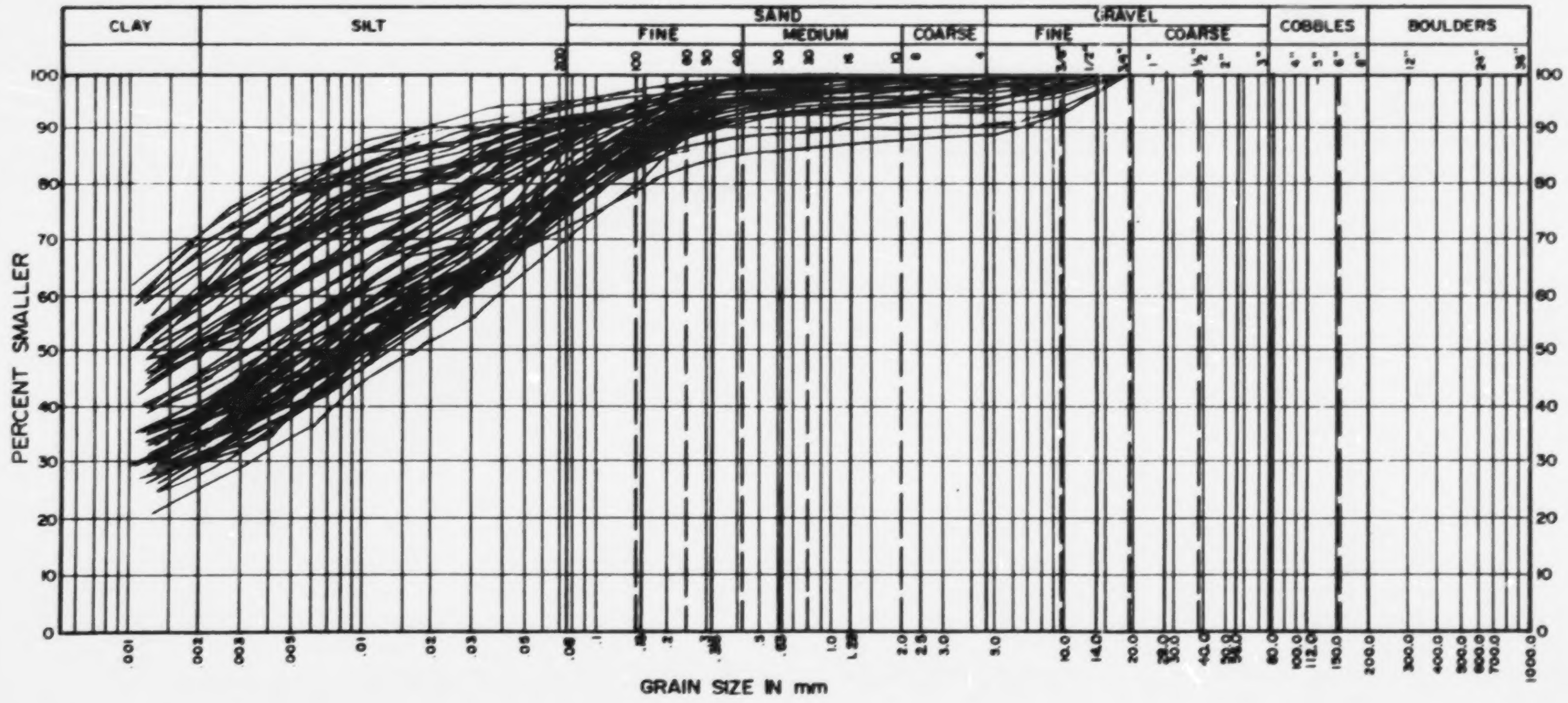
CLAY TILL (HIGHLY PLASTIC)



SILTY CLAY TILL (MEDIUM PLASTIC)



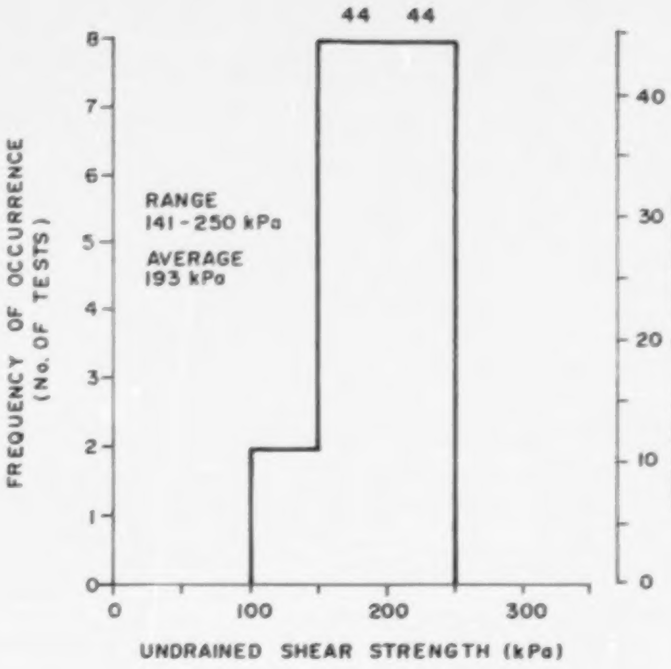
GLACIAL TILL



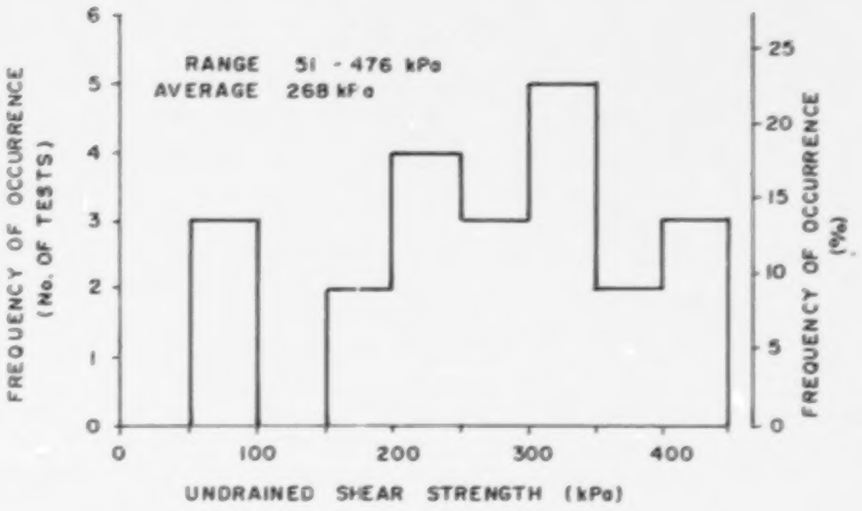
Alberta
OLDMAN RIVER DAM

LEFT BANK
GLACIAL TILL - I

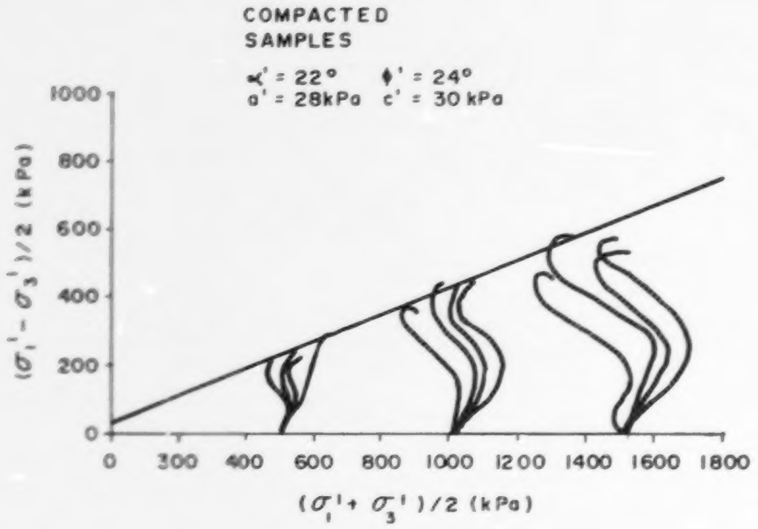
PLATE A-7

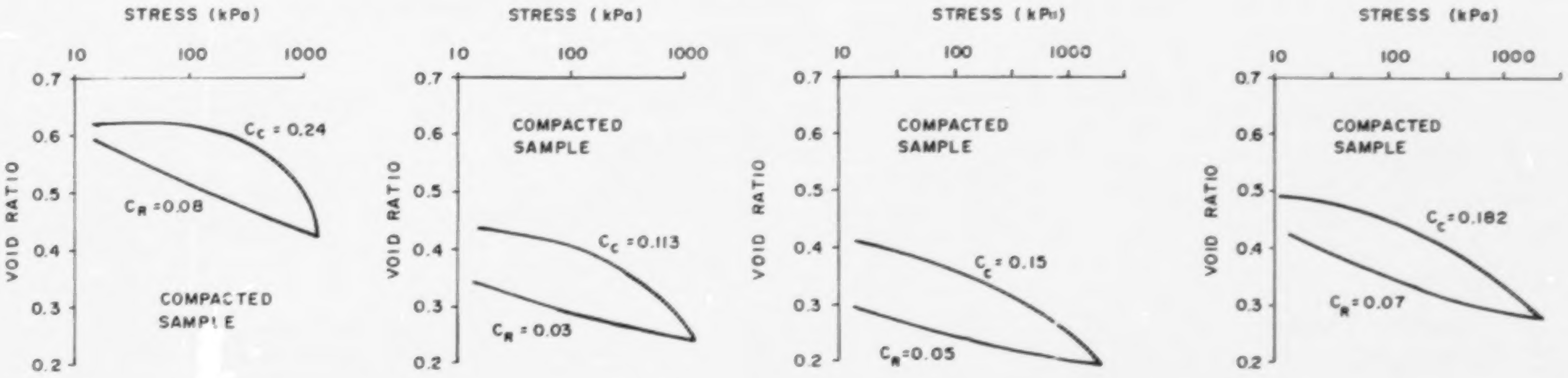


COMPACTED SAMPLES



UNDISTURBED SAMPLES



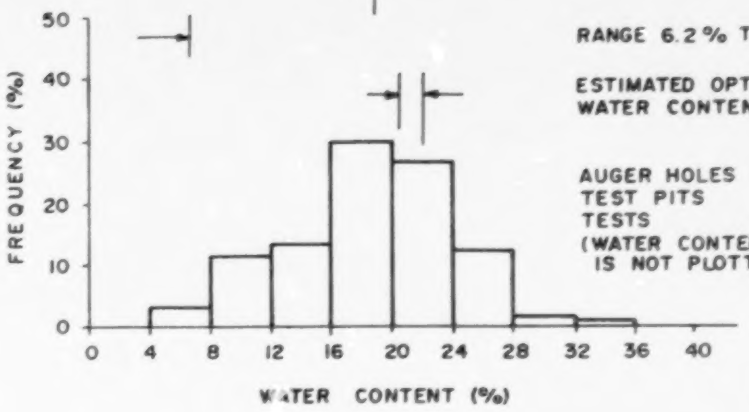


CONSOLIDATION RESULTS

Alberta uma
OLDMAN RIVER DAM

LEFT BANK
GLACIAL TILL - 2

PLATE A-8

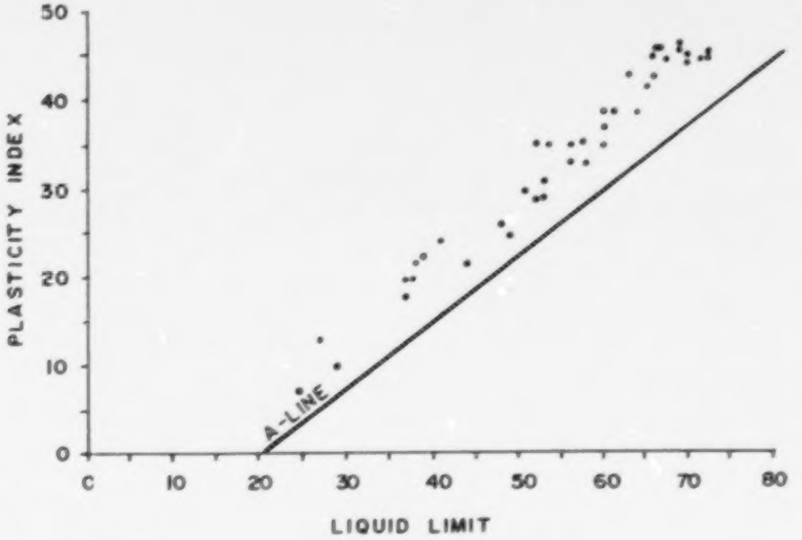


AVERAGE 18.6%

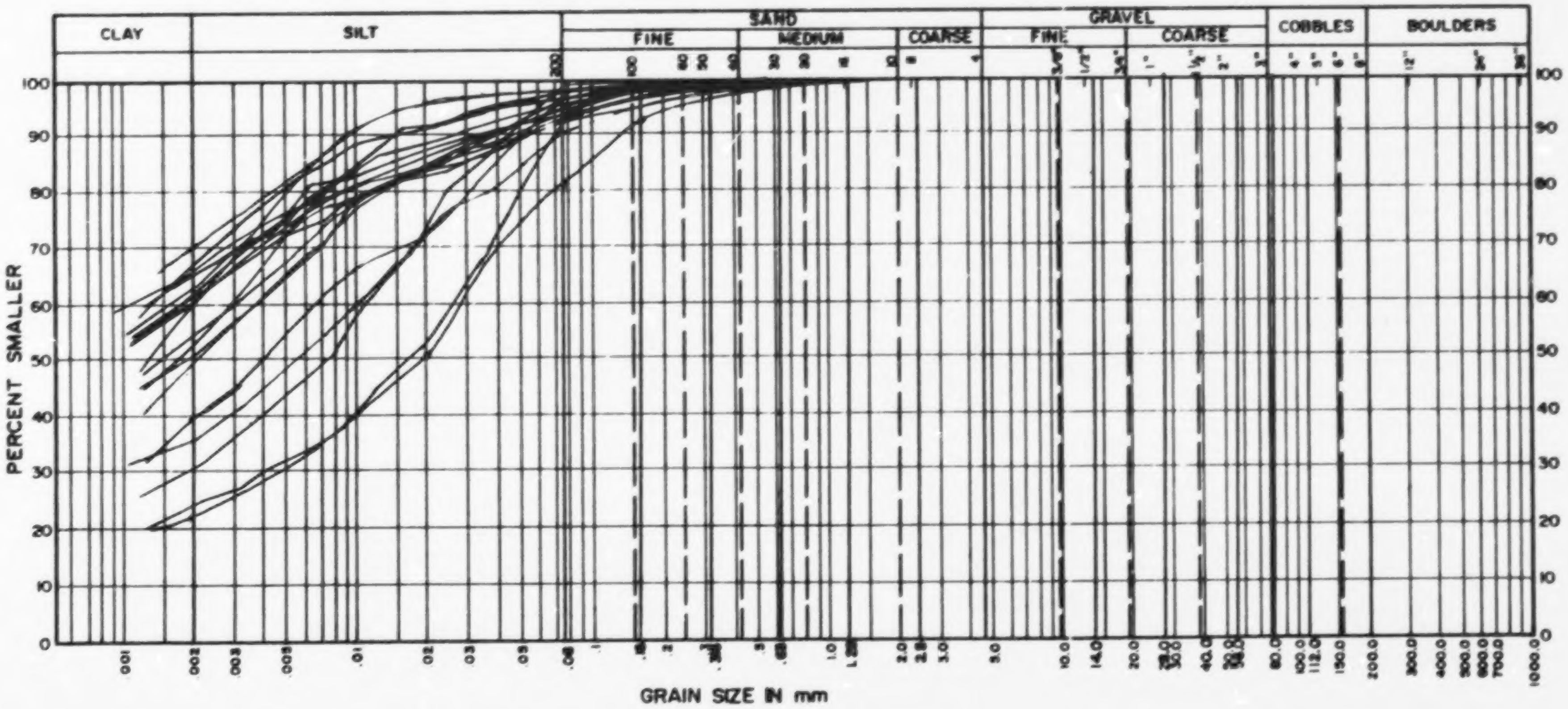
RANGE 6.2% TO 66.0%

ESTIMATED OPTIMUM
WATER CONTENT 20.2% TO 21.9%

AUGER HOLES = 27
TEST PITS = 5
TESTS = 172
(WATER CONTENT OF 66 %
IS NOT PLOTTED)



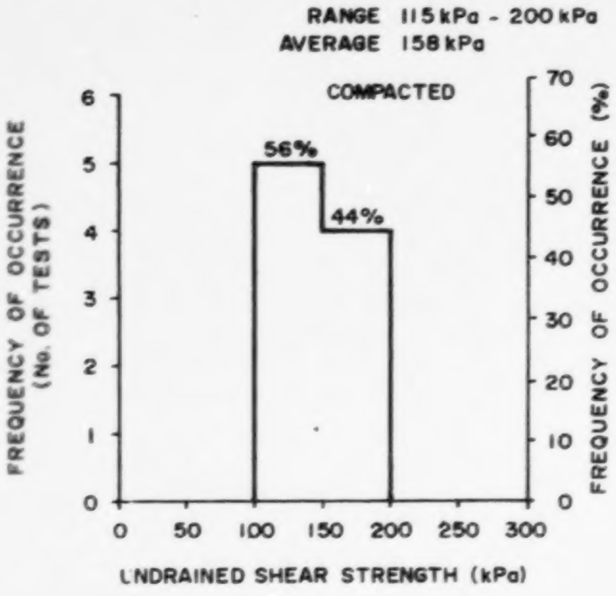
GLACIO - LACUSTRINE CLAY

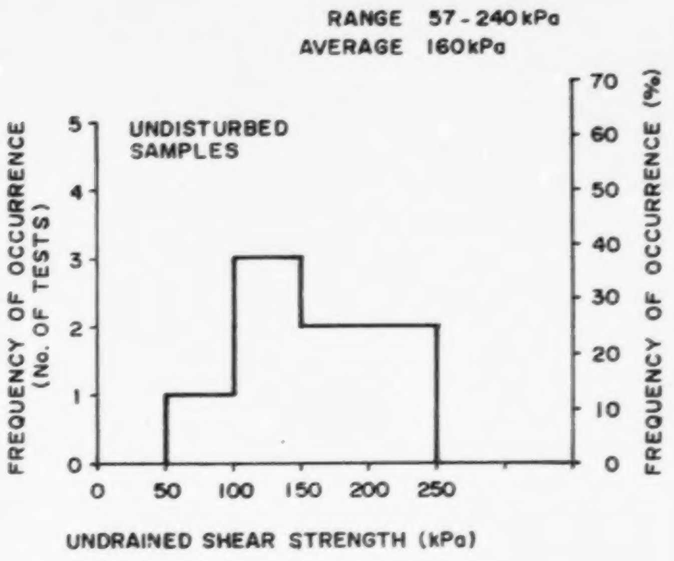


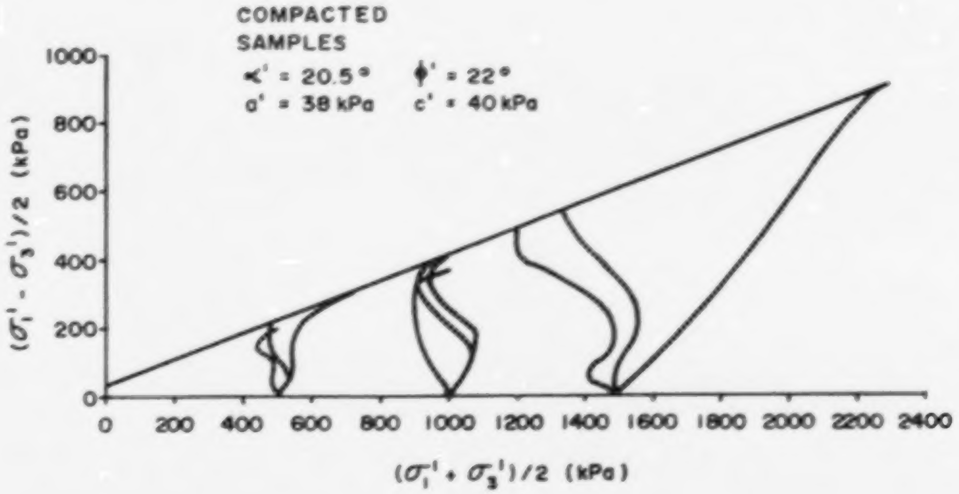
OLDMAN RIVER DAM

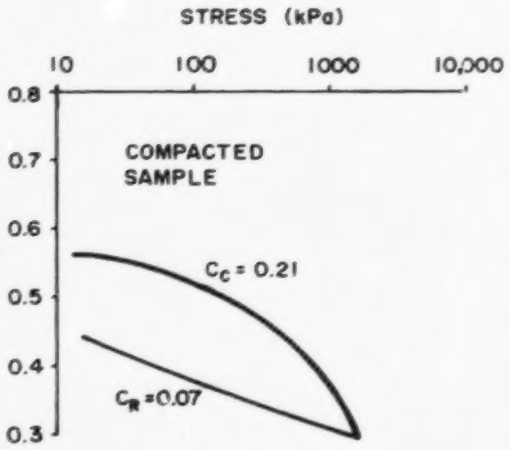
LEFT BANK
GLACIO-LACUSTRINE CLAY-I

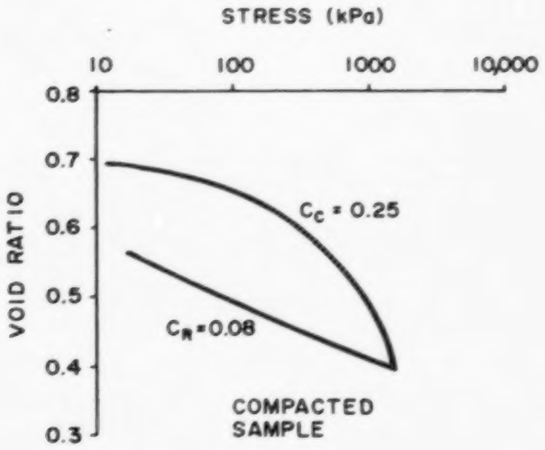
PLATE A-9







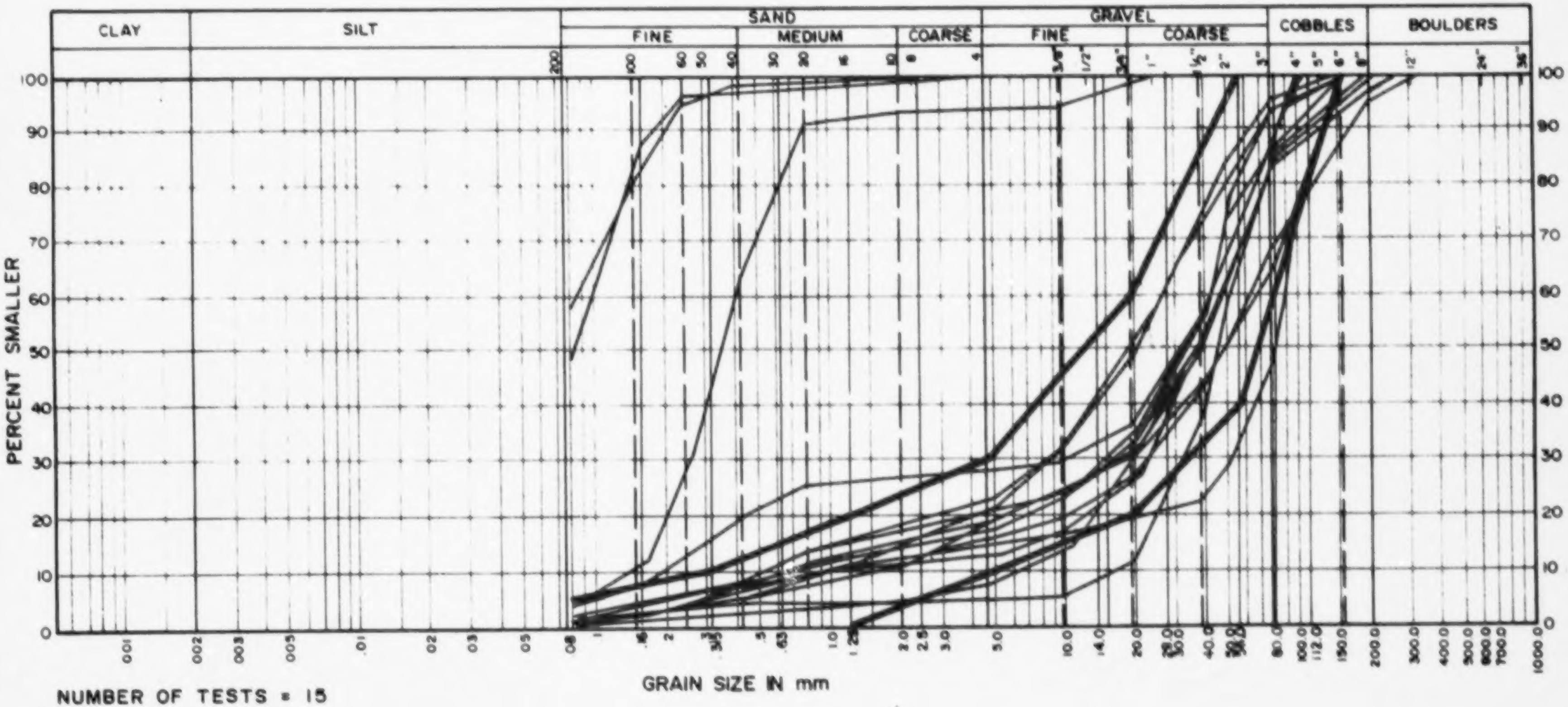




OLDMAN RIVER DAM

LEFT BANK
GLACIO-LACUSTRINE CLAY-2

PLATE A-10

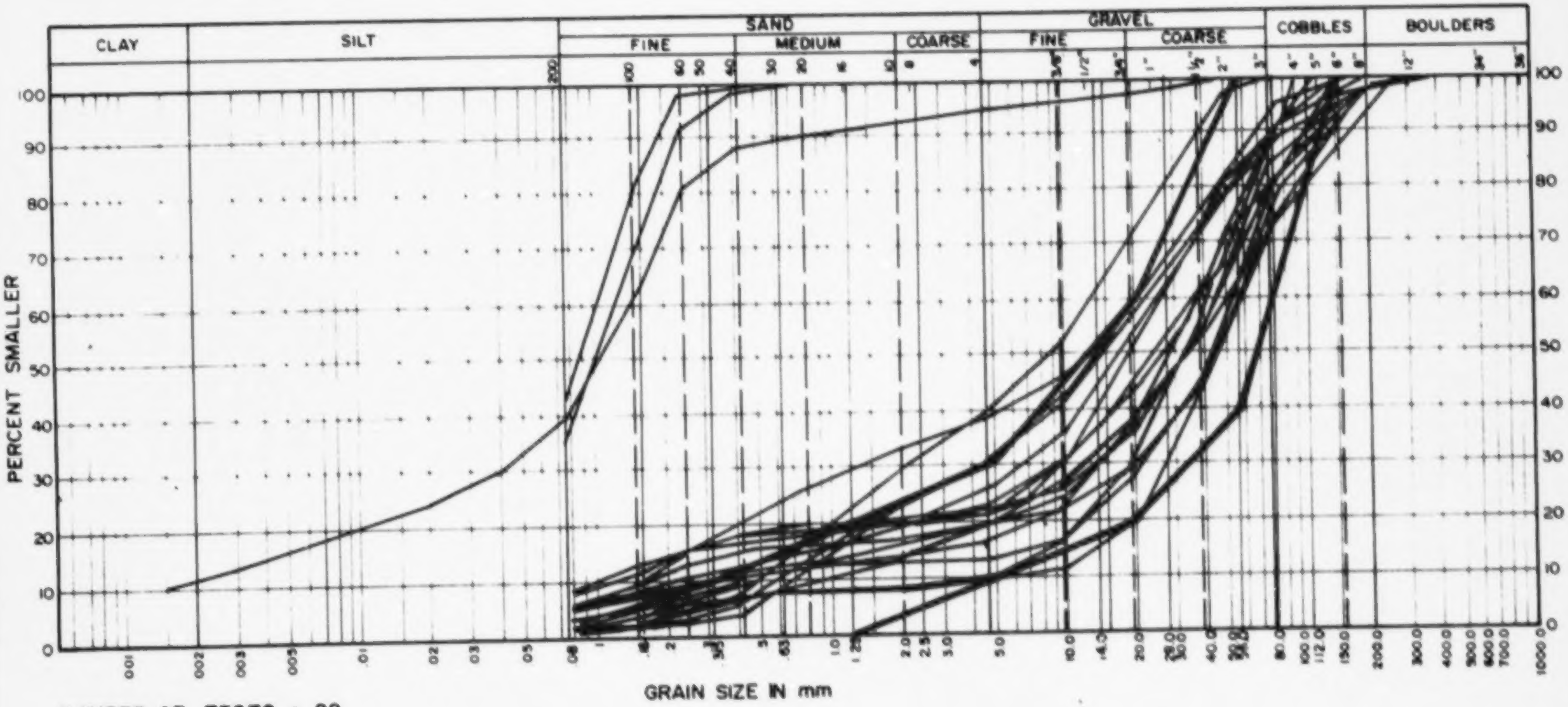


NUMBER OF TESTS = 15

Alberta
OLDMAN RIVER DAM

GRANULAR BORROW
AREA B1

PLATE A-11

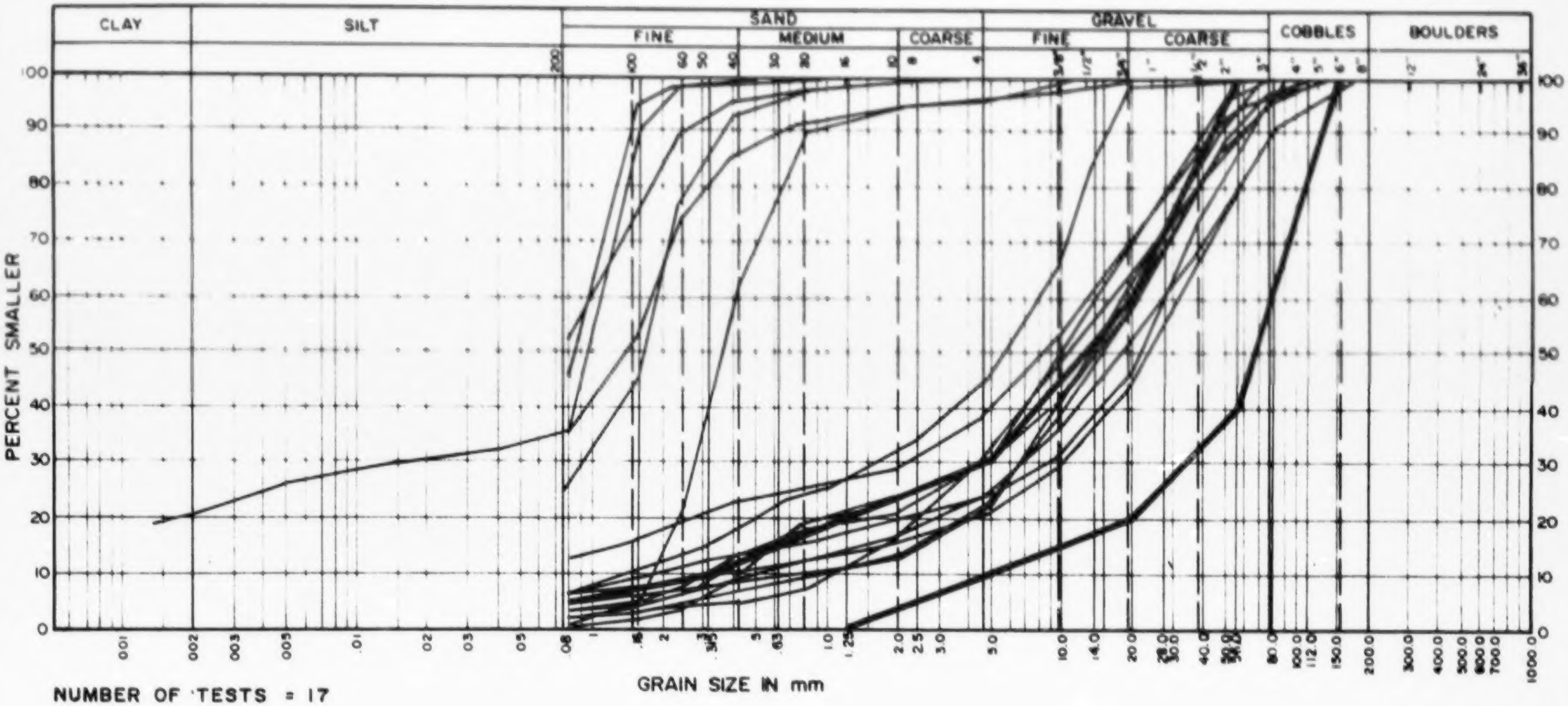


NUMBER OF TESTS = 20

Alberta
OLDMAN RIVER DAM

GRANULAR BORROW
AREA B2

PLATE A-12

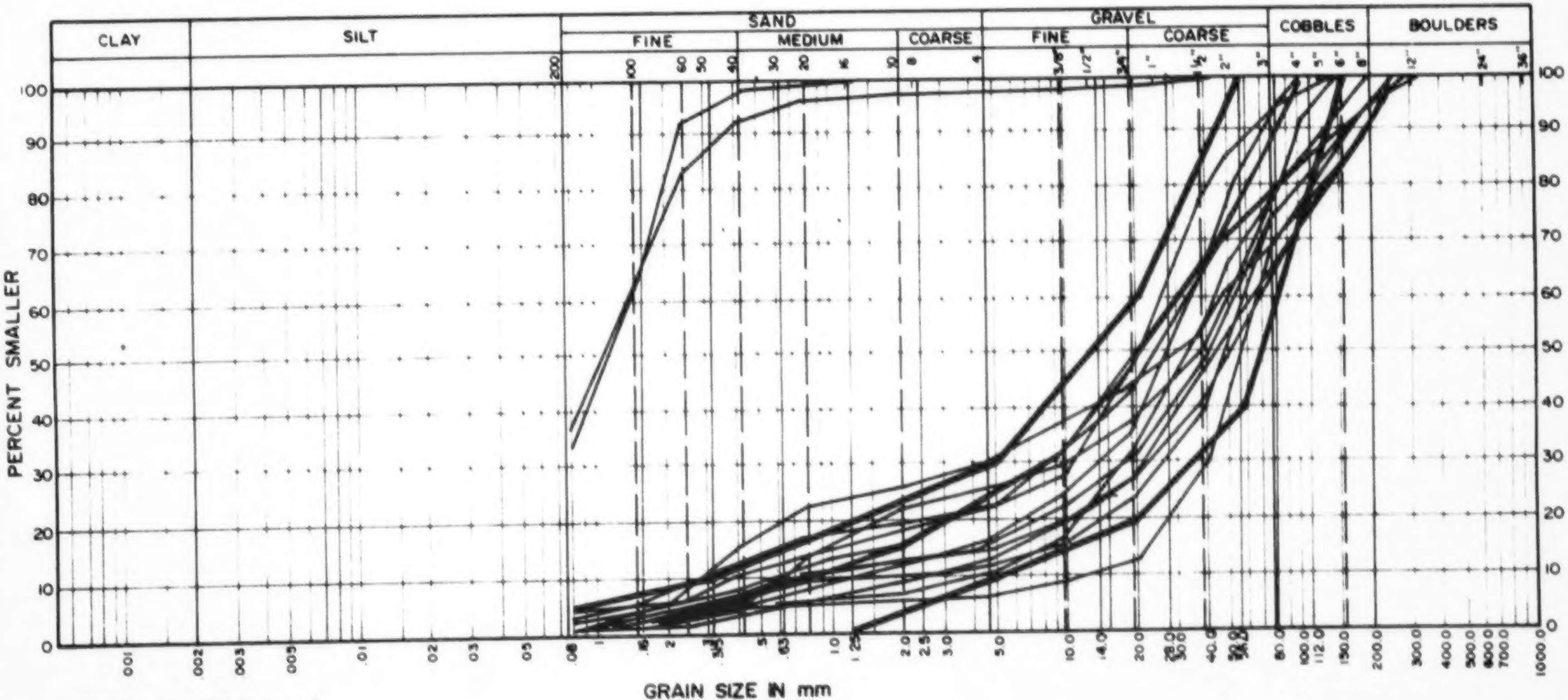


NUMBER OF TESTS = 17

Alberta ENVIRONMENT · UMA · OLDMAN RIVER DAM

GRANULAR BORROW
AREA B7

PLATE A-13

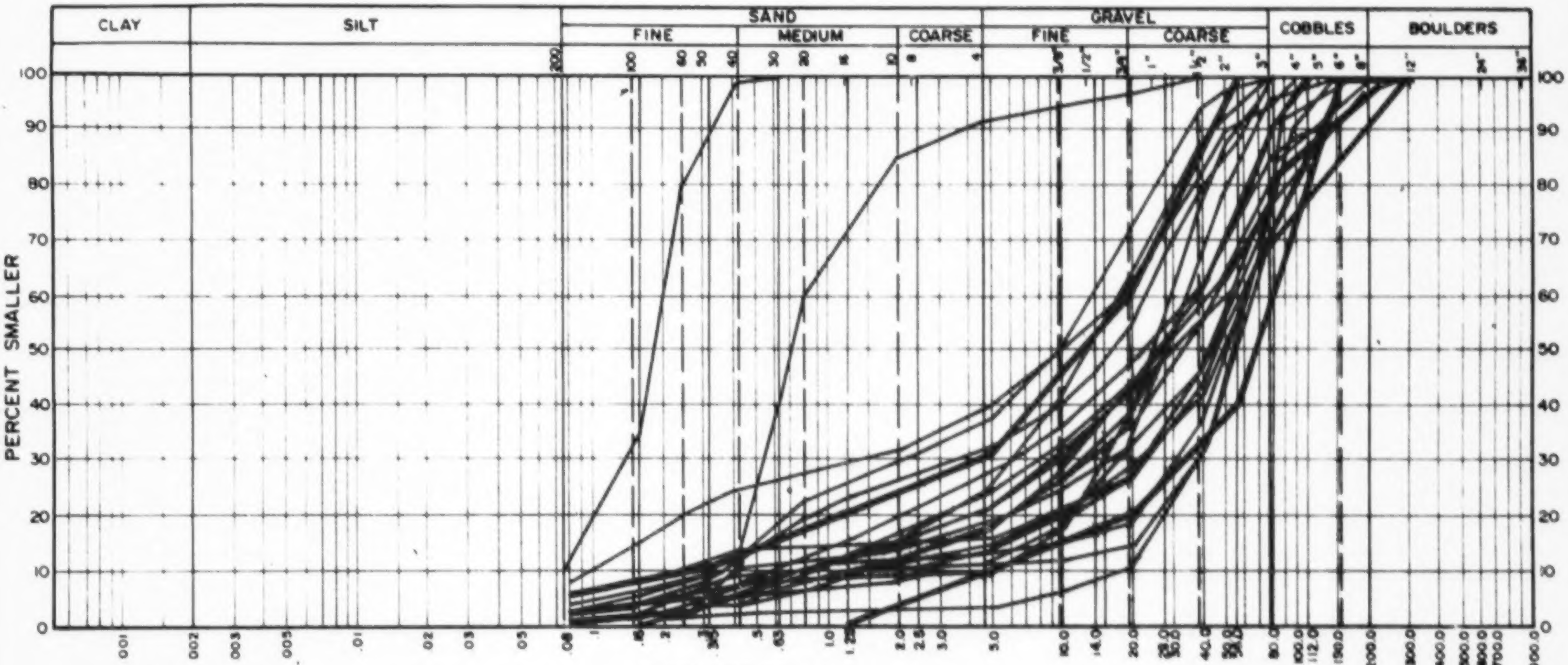


NUMBER OF TESTS = 16

OLDMAN RIVER DAM

GRANULAR BORROW
AREA B10

PLATE A-14

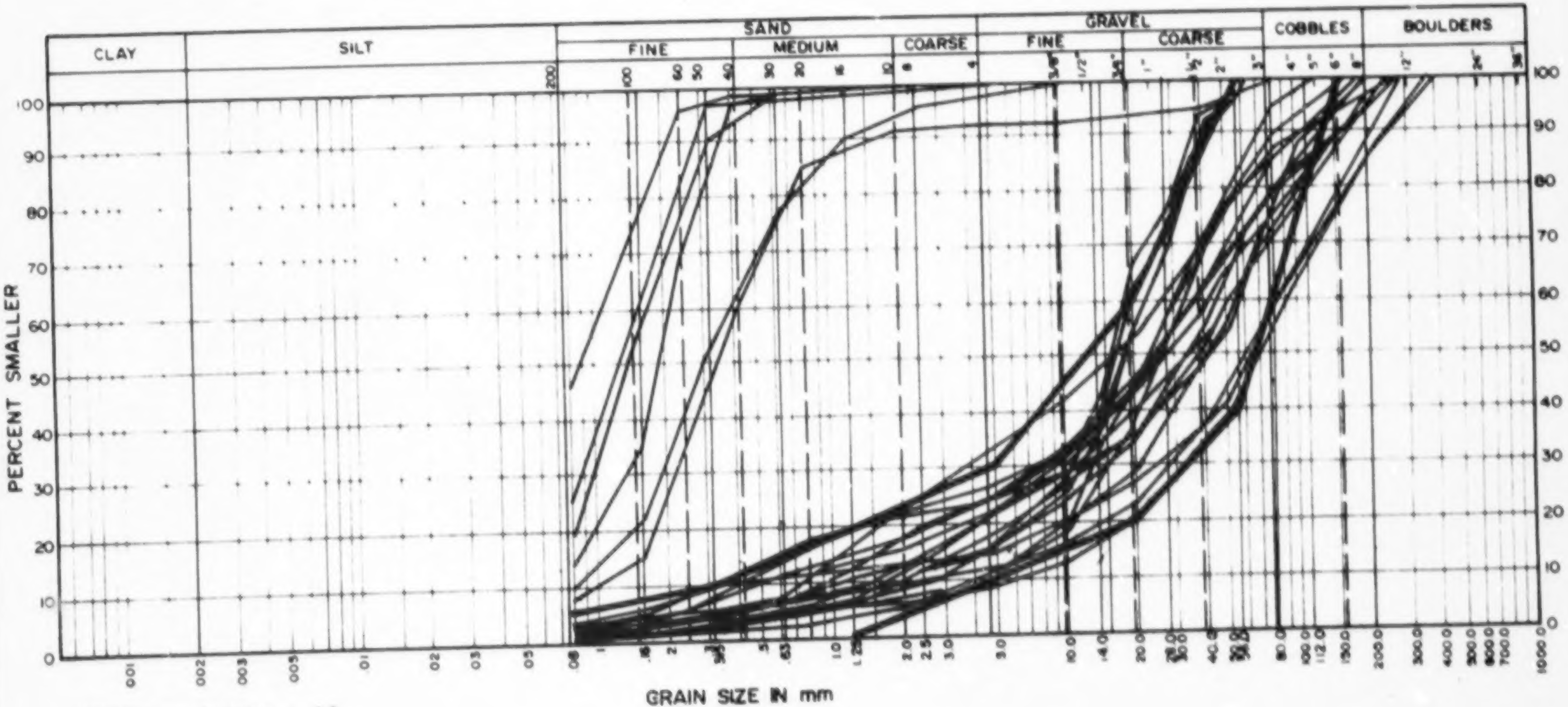


NUMBER OF TESTS = 22

OLDMAN RIVER DAM

GRANULAR BORROW
AREA B11

PLATE A-15

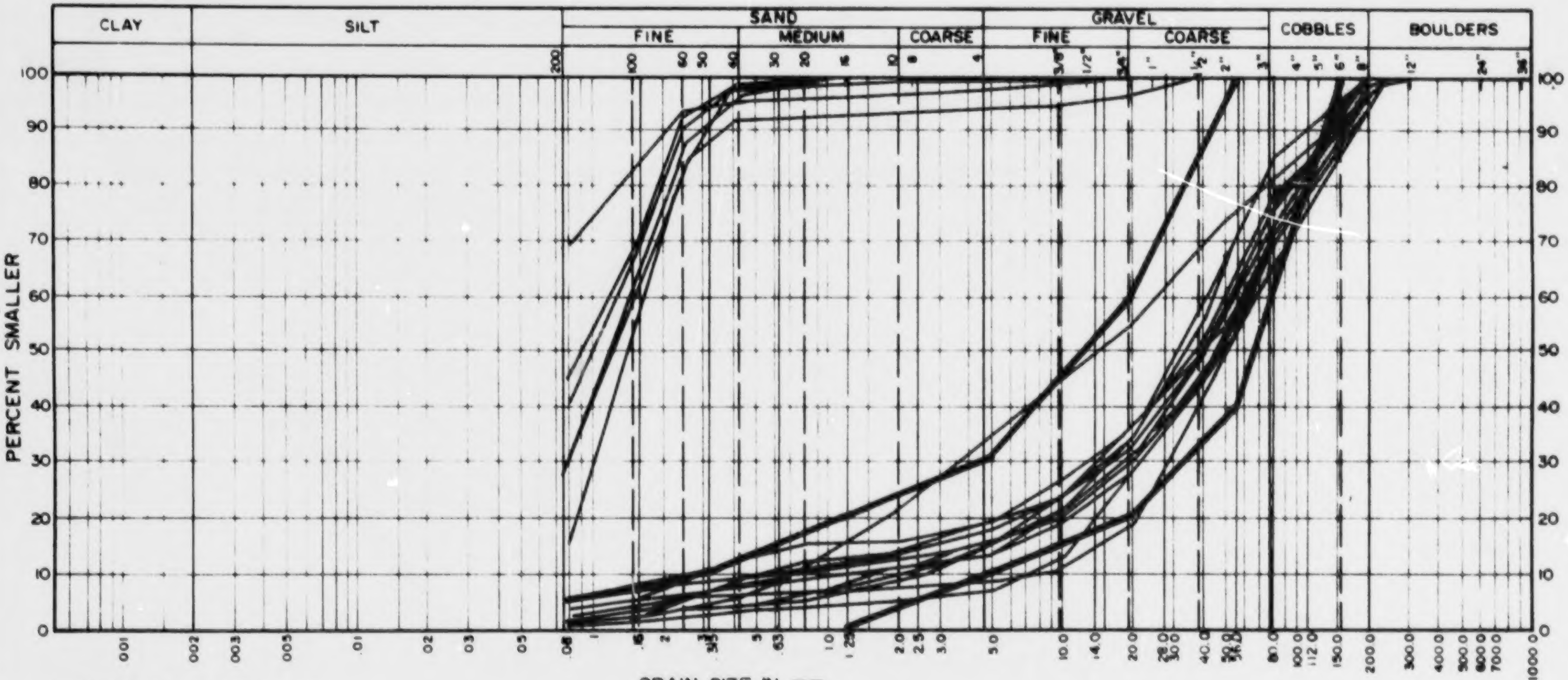


NUMBER OF TESTS = 26

OLDMAN RIVER DAM

GRANULAR BORROW
AREA B12

PLATE A-16

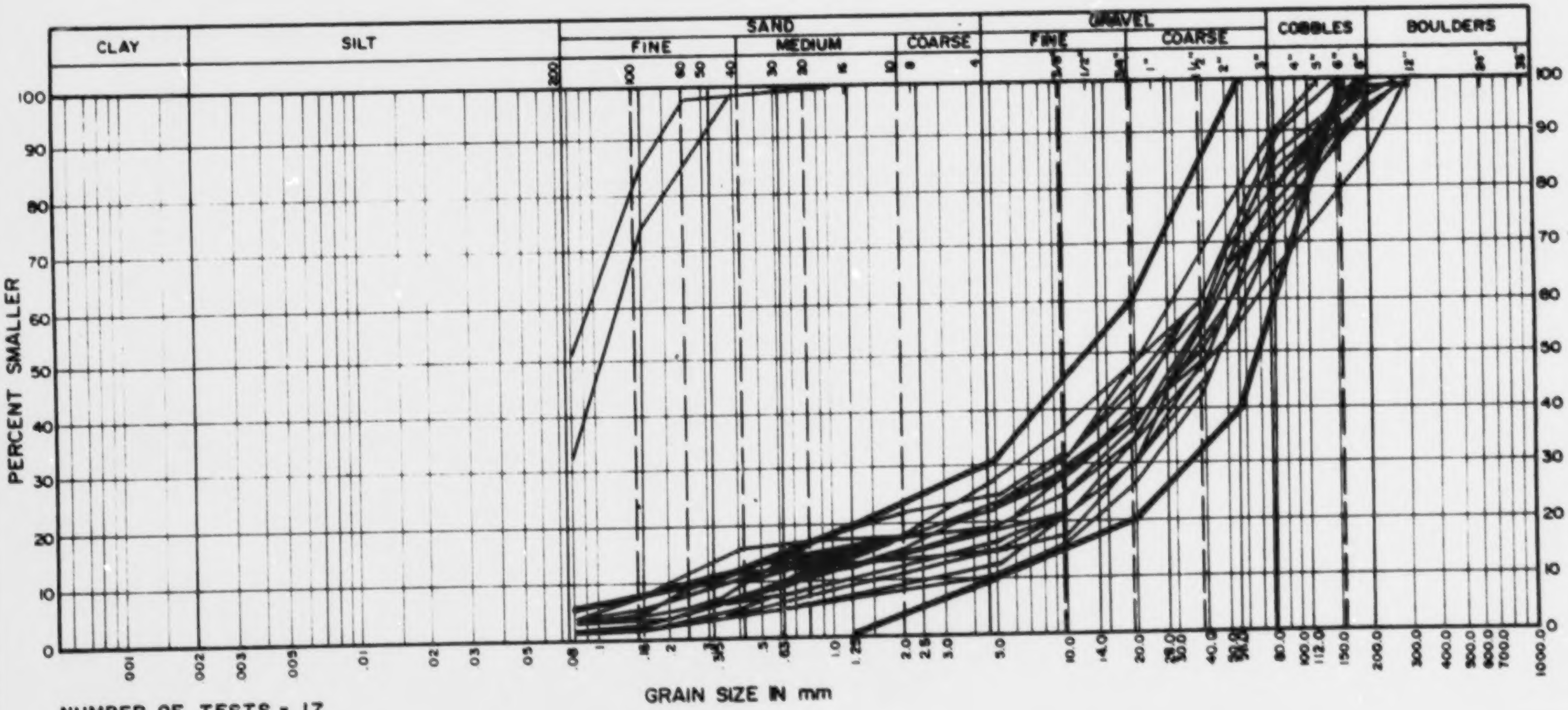


NUMBER OF TESTS = 20

OLDMAN RIVER DAM

GRANULAR BORROW
AREA B13

PLATE A-17

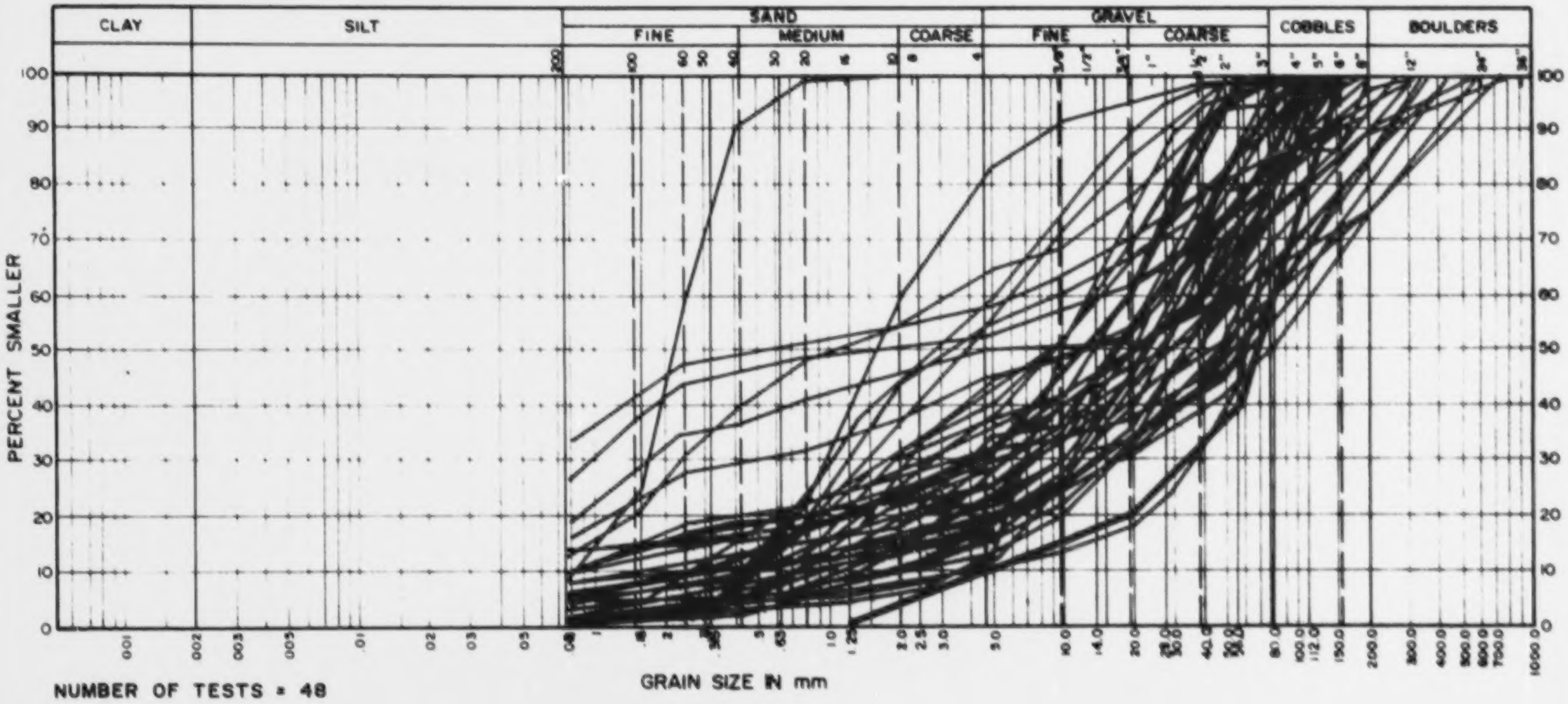


NUMBER OF TESTS = 17

OLDMAN RIVER DAM

GRANULAR BORROW
AREA B14

PLATE A-18



NUMBER OF TESTS = 48

OLDMAN RIVER DAM

GRANULAR BORROW
AREA B15

PLATE A-19